LOWER BRIDGE AT ENGLISH CENTER
Pennsylvania Historic Bridges Recording Project
Spanning Little Pine Creek at State Rt. 4001
English Center
Lycoming County
Pennsylvania

HAER No. PA-461

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HAER No. PA-461

Location:	Spanning Little Pine Creek at State Route 4001, English Center, Lycoming County, Pennsylvania.
USGS Quadrangle:	English Center, Pennsylvania (1973).
UTM Coordinates:	18/308740/4589330
Date of Construction:	1891.
Designer:	Dean and Westbrook, Engineers and Contractors (New York, New York).
Builder:	Dean and Westbrook, Engineers and Contractors (New York, New York).
Present Owner:	Pennsylvania Department of Transportation.
Present Use:	Vehicular bridge.
Significance:	The Lower Bridge at English Center is the only remaining bridge of two completed in 1891 to provide access to this once-important regional logging and tanning center. One of few surviving short-span roadway suspension bridges in Pennsylvania, the Lower Bridge is also an unusual variant of a braced-chain bridge. The bridge was listed in the National Register of Historic Places in 1978.
Historian:	Dr. Mark M. Brown, August 1997.
Project Information:	This bridge was documented by the Historic American Engineering Record (HAER) as part of the Pennsylvania Historic Bridges Recording Project - I, co-sponsored by the Pennsylvania Department of Transportation (PennDOT) and the Pennsylvania Historical and Museum Commission during the summer of 1997. The project was supervised by Eric DeLony, Chief of HAER.

1. Description

The Lower Bridge at English Center has a clear span of 300'-0", consisting of twelve panels each 25'-0" long.[1] It crosses Little Pine Creek on an approximately north-south alignment. The suspension chain is composed of pinned eye-bars which, like the entire bridge, are presumably of steel. The towers and the vertical members connecting the chain with the deck are built up from rolled sections. Diagonal tension members connect the eye-bars with the bottoms of the stiffening girders that support the deck. I-beams and open-grid steel decking make up the remainder of the cambered deck. Stone abutments and concrete anchorages support the entire structure. The north anchorage is 87'-3" from the tower while the south is 88'-7". There is a low-water crossing immediately downstream of the bridge (see measured drawings).

The chains consist of four eye-bars, each 4" x 7/8", per panel. Since each panel is the same length, the eye-bar lengths vary with the chain's slope. Two pairs of short plates or links connect the chains to eye-bars embedded in the concrete anchorages. The 39'-6"-tall tapered towers are braced at their top and middle with horizontal members. Wind bracing crosses above each portal between the middle and top members. Two 16"-wide plates, four 5" x 3" angles, and riveted double lacing make up the towers. Vertical members connecting the chains to the stiffening girders are pin-connected at the top and riveted at the bottom. The vertical members closest to the towers are built up from four 2" x 2" angles single-laced on all sides to form a square cross-section. Almost identical components are used on the other vertical members, except that the cross-section forms an "I". The diameters of diagonal tension members connecting the eye-bars with the bottoms of the stiffening girders range from 3/4" to 1-1/4", generally increasing toward mid-span. Several of these diagonal members are replacements, including those attached to the towers, the 1"-diameter rods at U0-L1 and L11-U12. Fragments of the original 3/4"-diameter rods were found in the tower pin connections. The replacements were attached by welding each rod's upper end to plates with hexagonal holes, which fit over the nut on the end of the pin.

The one-lane bridge has 15'-0" of clear roadway. The deck consists of open-grid steel decking resting on 7"-deep stringers (I-beams except the outermost, which are C-sections) with 12"-deep deck beams. Four 3" x 2" angles are riveted to a vertical 24" plate to make up the I-section stiffening girder. Lower lateral bracing in a single Warren pattern completes the deck. The open-grid road surface and most of the deck system are not original. Some of the diagonal tension members have been replaced over the years. In addition, extensive repairs have been made to the plates which connect the tension members to the stiffening girders.

The stone masonry abutments have courses approximately 12" high. Decorative cast-iron plates above each portal identify the contractors and the county commissioners.

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[1] All dimensions are based on field measurements made by HAER architects, June 1996, and A. G. Lichtenstein and Associates, "BMS No. 41-4001-0270-0000, Final Bridge Inspection Report," November 1994 (bridge inspection files, PennDOT Maintenance District 3-2, Montoursville, Pennsylvania).

2. History

Located in the flats of the steep-walled Little Pine Creek valley, English Center, once spelled "Centre," was a vibrant logging and tanning community during the second half of the nineteenth century.[2] The community probably existed well before it received a post office in 1844 and was named for the many members of the English family living in the area. The lumberjacks worked through winter so that they could use spring floods to carry the logs down Little Pine Creek and on to Williamsport via the Susquehanna River. A very profitable byproduct of the lumber industry was hemlock bark. The bark was transported by wagon to tanneries such as the one at the east end of English Center. Coal from modest deposits in the area was also transported to town to fuel the tannery.[3]

While the spring freshets were central to the transport of timber to market, heavy snow and rainfall in narrow vallcys denuded of trees were also a serious hazard. On 1 Junc 1889, a tremendous flood struck Lycoming County. Writing threc ycars later, one local historian observed that the flood tore through thc Little Pine Creck valley

> with terrific force, destroying fine bottom farm lands by covering them with sand and stones, sweeping away fences, bridges, mills, and houses, leaving utter desolation behind.[4]

Elsewhere, the historian also observed that "the water came down the narrow ravine in which [English Center] is situated in a mighty torrent, filling it from hill to hill, and the inhabitants were forced to fly to safety."[5]

In the years following the great flood, Lycoming County spent hundreds of thousands of dollars on bridges.[6] English Center received two replacement bridges: the Lower Bridge and the Upper Bridge, the latter of which has been demolished. While eounty records are incomplete, they clearly show that the process of replacing the bridges was not smooth. Citizens from English Center petitioned the Quarter Session Court on several occasions that viewers, or road surveyors, be appointed to locate new bridges, only to have yet other residents object to the viewers' findings. In addition, the bridge contractors seem to have had difficulty receiving

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[2] See "English Center," in Beach Nichols, *Atlas of Lycoming County* (Philadelphia: A. Pomeroy and Company, 1873), 38.

[3] John F. Meginness, ed., *History of Lycoming County, Pennsylvania* (Chicago: Brown, Runk and Company, 1892), 687-92; and Harry Stephenson, Sr., *History of Little Pine Valley* (Camp Hill, Pennsylvania: self-published, 1992), 34, 51-54, 61-66.

[4] Meginness, *History of Lycoming County*, 688.

[5] Meginness, *History of Lycoming County*, 692.

[6] Meginness, *History of Lycoming County*, 319-20. The same storm system that caused the 1 June 1889 flood was also responsible for the tragic Johnstown, Pennsylvania, flood.

payment. These records are of interest because both the original petitions and the viewers' reports survive and consequently provide a rare view into one aspect of nineteenth-century bridge construction.

On 14 August 1889, eleven citizens from English Center, including two members of the English family, petitioned the Quarter Session Court "that a view be appointed to vacate the two old bridge sites destroyed by June flood at English Centre Pa. and to locate one or more sites at English Centre."[7] The court agreed and appointed three viewers who reported back to the court in early September (see Figure 1). After viewing the damage, the viewers recommended abandoning the existing bridge sites on either end of town. In place of the old bridges, they proposed building a new bridge at the store that was near the middle of town. They also proposed a new road along the south, or left, bank of Little Pine Creek to connect with the public roads at the old bridge sites. The report included formulaic language to the effect that the improvements were too expensive for the township and that the county should build the bridge. Consequently, the county commissioners estimated the cost for the proposed bridge to be $9,000.

Two objections were raised. One was on procedural grounds and the other, signed by at least seventy-six people, objected to the findings of the viewers.[8] The court responded by appointing new people to review the matter. The re-viewers concluded that a new 384'-6"-long bridge should be built near the old bridge site at the tannery upstream of town (see Figure 2), that the proposed bridge near the store would be inconvenient and dangerous, and that a bridge site not mentioned in the court order, two miles downstream, be abandoned. No mention was made of the site of the other bridge, at the lower end of town, damaged by the flood. The commissioners estimated that this proposal would cost $25,000.[9]

These efforts apparently did not resolve the controversy. On 21 June 1890, yet another exception was filed. This time the objection was that the re-viewers were not authorized to comment on vacating the site two miles downstream from English Center. The Court agreed and for a second time set aside the report. What happened next can only be inferred, as the Commissioners' *Minutes* for 1889 and 1890 are lost. It appears that a decision was forced, or some sort of compromise was reached, for on October 2, the attorneys in the affair reported to Judge John J. Metzger that they had settled the matter. Dean and Westbrook, contractors and

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[7] This section is based on Lycoming County, Pennsylvania, *Quarter Session Court Dockets,* No. 20 (September 1889); or a copy of the petition quoted see also Lycoming County, *Road Minutes,* 9:60 (both: Office of Prothonotary, Lycoming County Courthouse, Williamsport, Pennsylvania).

[8] There is a confusing point in the documents as both objections were formally filed with the court after the court appointed the review. One possible explanation is that the objections were raised in a timely and appropriate manner but were not filed with the court until required by later legal maneuvering.

[9] Lycoming County, *Quarter Session Court Dockets,* No. 10 (December 1889).

bridge engineers, were issued contracts on 25 August, 11 October, and 27 December 1890, by Commissioners Abner M. Foresman, T. J. Strebeigh, W. S. Starr.[10]

New commissioners, however, were in authority by mid-January 1891. The following entries appear in the Commissioners's *Minutes*:

> Examined the Bridge Contract for the Englishtown [*sic*] Bridge.[11]
>
> The Board took up the English Centre Bridge Contract and examined the specifications closely. When they adjourned it was agreed to go to English Centre and see if in their judgment the contract had been filled to warrant a payment of $13,000 as demanded.[12]
>
> Heard some citizens of Pine Township in regard to the lower bridge at English Centre.[13]
>
> By advice of their attomey [the Commissioners] answered Mr. Dean of the firm of Dean and Westbrook Bridge Contractors by demanding a Bond in the sum of Thirty thousand Dollars. Mr. Dean left without saying whether it would be furnished.[14]
>
> Willard English presented his claim for keeping in repair the Temporary Bridge at English as per verbal contract between the County Commissioners of Lycoming County and Willard English and A. J. Jones. An order for the amount — $40 — was drawn.[15]

In May, and again in July, the commissioners went to English Center to examine the bridge, or bridges — both terms are found in the minutes. By September the commissioners were petitioning the Quarter Session Court for permission to settle in full for both, presumably Upper and Lower, bridges. The court granted the request, but not before the viewers "recommended a reduction in the aggregate of Twelve thousand three hundred ninety dollars for a reduction of the aggregate of Ten Thousand dollars." Unfortunately, in the absence of the contract details, the precise cost of the bridges remains uncertain. Nor was this the end of the matter, for the

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[10] The National Register form contains the misspelling "Dean & Westerbrooke." Lycoming County, *Quarter Session Court Dockets*, No. 10 (December 1889); Lycoming County, *Road Minutes*, 9:62.

[11] Lycoming County, *Minutes of the Lycoming County Commissioners* (Recorder of Deeds Office, Lycoming County Courthouse, Williamsport, Pennsylvania; hereinafter cited as *Minutes*), 3 (17 January 1891).

[12] Lycoming County, *Minutes*, 4 (21 January 1891).

[13] Lycoming County, *Minutes*, 4 (21 January 1891).

[14] Lycoming County, *Minutes*, 6 (2 February 1891).

[15] Lycoming County, *Minutes*, 8 (9 February 1891).

commissioners went yet again to English Center the same month to determine how much fill was needed for the approaches to the bridges.[16]

Historic photographs of the English Center area reveal that the Upper Bridge was of the same type of construction as the Lower Bridge. The major distinction was that the Upper Bridge was larger: sixteen panels can be counted on the Upper Bridge versus the twelve panels on the Lower Bridge. A replacement for the Upper bridge was built in 1932.[17] The Lower Bridge at English Center was placed on National Register in 1978.

3. The Bridge Contractor

Dean and Westbrook was formed about 1870. In 1883, partners C. W. Dean and John A. Westbrook moved to New York City from their initial headquarters in Cleveland, Ohio, because much of the practice was centered on the mid-Atlantic and adjoining states. Dean and Westbrook were agents for the Phoenix Bridge Company of Phoenixville, Pennsylvania. Between 1885 and 1893 Dean and Westbrook ordered seventy-eight bridges from Phoenix Bridge for Pennsylvania clients alone. A preliminary search of corporate records indicates, however, that Phoenix Bridge did not fabricate the English Center bridges.[18] Since noted bridge historian Victor C. Darnell reported that Dean and Westbrook began advertising under their own name after 1891, it may be that the pair actually made their break with the English Center bridges.[19]

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[16] Lycoming County, *Minutes*, 26 (18 and 19 May 1891); 34 (6 and 7 July 1891); 49 (7 September 1891); 51 (21 September 1891); and 52 (22 and 23 September 1891). As of this writing, microfilm of the *Quarter Session Court Dockets* for 1891 concerning two 1891 bridge inspections was unavailable; see No. 15 (1891): 467, and No. 16 (1891): 17 (microfilm roll 236, Office of Prothonotary, Lycoming County Courthouse, Williamsport, Pennsylvania).

[17] For photographs of the Upper Bridge, see Stephenson, *History of Little Pine Valley*, 63-64, 76, 83. The photograph on p. 74 is probably the Lower Bridge.

[18] U.S. Department of the Interior, Historic American Engineering Record (HAER) No. PA-98, "Washingtonville Bridge," 1986, p. 2; and HAER No. PA-412, "Walnut Street Bridge," 1996, pp. 7-10 (both: Prints and Photographs Division, Library of Congress, Washington, D.C.). Margaret McNinch (Archives and Manuscripts, Hagley Museum and Library, Greenville, Delaware) searched the "History of Orders" Books of the Phoenix Bridge Company and found no reference to the English Center bridges or to any new bridge construction in Lycoming County. Phoenix did perform repairs on a 135'-0" through span on Larrys Creek in Lycoming County in January 1891 (Margaret McNinch, conversation with author, 4 August 1997). For more on Phoenix Bridge see Thomas R. Winpenny, *Without Fitting, Filing, or Chipping: An Illustrated History of the Phoenix Bridge Company* (Easton, Pennsylvania: Canal History and Technology Press, 1996).

[19] Victor C. Darnell, to author and Blythe Semmer, 30 July 1997.

4. Hybrid Suspension-Truss System

The particular integration of a suspension system (eye-bar chains) with the truss system (diagonals and compression members) used at English Center is unusual. Trusses are among the most common systems used to stiffen suspension bridges against bending and oscillation of the deck. At English Center, however, latticed elements, visually resembling compression members, replace the vertical hanging cables. Furthermore, the diagonals do not connect the top and bottom chords of a separate truss, but rather they connect the deck to the main suspension elements.

The unusual structural system has generated a variety of ideas about how the bridge behaves. Darnell suggested that English Center might be a double cantilever. Pointing to the Queensboro Bridge over the East River in New York City, he observed that it does not have the center span typically associated with cantilever bridges. Might not English Center be a similar case? A further example is given by the Northampton Street Bridge spanning the Delaware River between Easton, Pennsylvania, and Phillipsburg, New Jersey. Built in 1895, the Northampton Street Bridge is a cantilever that deliberately used eye-bars to give the visual impression of a suspension bridge.[20]

On the other hand, consulting engineer Jackson Durkee, formerly of Modjeski and Masters, considered that the

> purpose of the X-bracing in each cable panel is to supply some stability to the structure. Certainly it does not serve as truss-type supporting structure in any sense. On a very long and slender bridge, such X-bracing might add a little torsional stability to the structure under wind loading; however, on this short-span bridge such torsional resistance is undoubtedly not at all necessary.[21]

Consequently, Durkee concluded that the cross-bracing is superfluous and that the bridge must therefore be a suspension bridge.

Dr. Dario A. Gasparini considered English Center to be what Steinman referred to as a "braced-chain" suspension system. He suggested that

> unlike the [Northampton Street] Bridge, the number of eyebars in the top chain of the English Center Bridge does not decrease from the towers to the midspan. Therefore the intent of the designer at [English Center] is unclear. Loads can be carried *either* by "suspension behavior" or "truss behavior." The stiffer mode will

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[20] Victor C. Darnell, to PennDOT Maintenance District 3-2, 29 January 1995 (bridge inspection files, PennDOT Maintenance District 3-2, Montoursville, Pennsylvania); Carol P. Henry, "Northampton Street Bridge," Northampton County, Pennsylvania, National Historic Civil Engineering Landmark Nomination, 1981, American Society of Civil Engineers, Reston, Virginia. See HAER No. NY-19 for documentation of the Queensboro Bridge.

[21] Jackson L. Durkee, to Vance P. Packard (Office of Historic Preservation), 11 September 1978 (bridge inspection files, PennDOT Maintenance District 3-2, Montoursville, Pennsylvania).

dominatc. Structural analysis performed by A. G. Lichtenstein for PennDOT indicates that the bridge carries loads mostly as a truss.[22]

While short-span suspension bridges are increasingly rare in Pennsylvania, the unusual structural system adds additional distinction and importance to the Lower Bridge at English Center. Given that the designer's intentions are unclear, the Lower Bridge represents an opportunity for further structural analysis of nineteenth-century American suspension bridge engineering.

[22] Dr. Dario A. Gasparini (professor of civil engineering, Case Western Reserve University), to author, 4 August 1997; STAAD-III output, 22 July 1994, in A. G. Lichtenstein and Associates, "BMS No. 41-4001-0270-0000." On "braced-chain" suspension bridges see David B. Steinman, *A Practical Treatise on Suspension Bridges, Their Design, Construction and Erection* (New York: John Wiley and Sons, 1929), 103-10. Examples of "braced-chain" suspension bridges include the 1877 Point Bridge in Pittsburgh, Gustav Lindenthal's 1922 proposal for a Hudson River bridge in New York, and the Grand Avenue Viaduct in St. Louis.

SOURCES CONSULTED

Darnell, Victor C. To PennDOT Maintenance District 3-2, 29 January 1995. Bridge inspection files, PennDOT Maintenance District 3-2, Montoursville, Pennsylvania.

______. To author and Blythe Semmer, 30 July 1997.

Durkee, Jackson L. To Vance P. Packard, Pennsylvania Office of Historic Preservation, September 11, 1978. Bridge inspection files, PennDOT Maintenance District 3-2, Montoursville, Pennsylvania.

Gasparini, Dario A. To author, 4 August 1997.

Henry, Carol P. "Northampton Street Bridge," Northampton County, Pennsylvania. National Historic Civil Engineering Landmark Nomination, 1981. American Society of Civil Engineers, Reston, Virginia.

A. G. Lichtenstein and Associates. "BMS No. 41-4001-0270-0000, Final Bridge Inspection Report," November 1994. Bridge inspection files, PennDOT Maintenance District 3-2, Montoursville, Pennsylvania.

Meginness, John F., ed. *History of Lycoming County, Pennsylvania.* Chicago: Brown, Runk and Company, 1892.

Stephenson, Harry, Sr. *History of Little Pine Valley.* Camp Hill, Pennsylvania: self-published, 1992.

Lycoming County, Pennsylvania. *Minutes of the Lycoming County Commissioners.* Recorder of Deeds Office, Lycoming County Courthouse, Williamsport, Pennsylvania.

______. *Quarter Session Court Dockets.* Office of Prothonotary, Lycoming County Courthouse, Williamsport, Pennsylvania.

______. *Road Minutes.* Office of Prothonotary, Lycoming County Courthouse, Williamsport, Pennsylvania.

McNinch, Margaret. Conversation with author, 4 August 1997.

Pennsylvania Department of Transportation. Bridge inspection file, BMS No. 41-4001-0270-0000. PennDOT District 3-0, Montoursville, Pennsylvania.

Steinman, David B. *A Practical Treatise on Suspension Bridges, Their Design, Construction and Erection.* New York: John Wiley and Sons, 1929.

U.S. Department of the Interior, Historic American Engineering Record (HAER) No. PA-98, "Washingtonville Bridge," 1986. Prints and Photographs Division, Library of Congress, Washington, D.C.

______, HAER No. PA-412, "Walnut Street Bridge," 1996. Prints and Photographs Division, Library of Congress, Washington, D.C.

Winpenny, Thomas R. *Without Fitting, Filing, or Chipping: An Illustrated History of the Phoenix Bridge Company.* Easton, Pennsylvania: Canal History and Technology Press, 1996.

APPENDIX: Figures

Figure 1 "Map from Report of Viewers," 4 September 1889, from Lycoming County, *Quarter Session Court Dockets*, No. 20 (September 1889) (Office of Prothonotary, Lycoming County Courthouse, Williamsport, Pennsylvania).

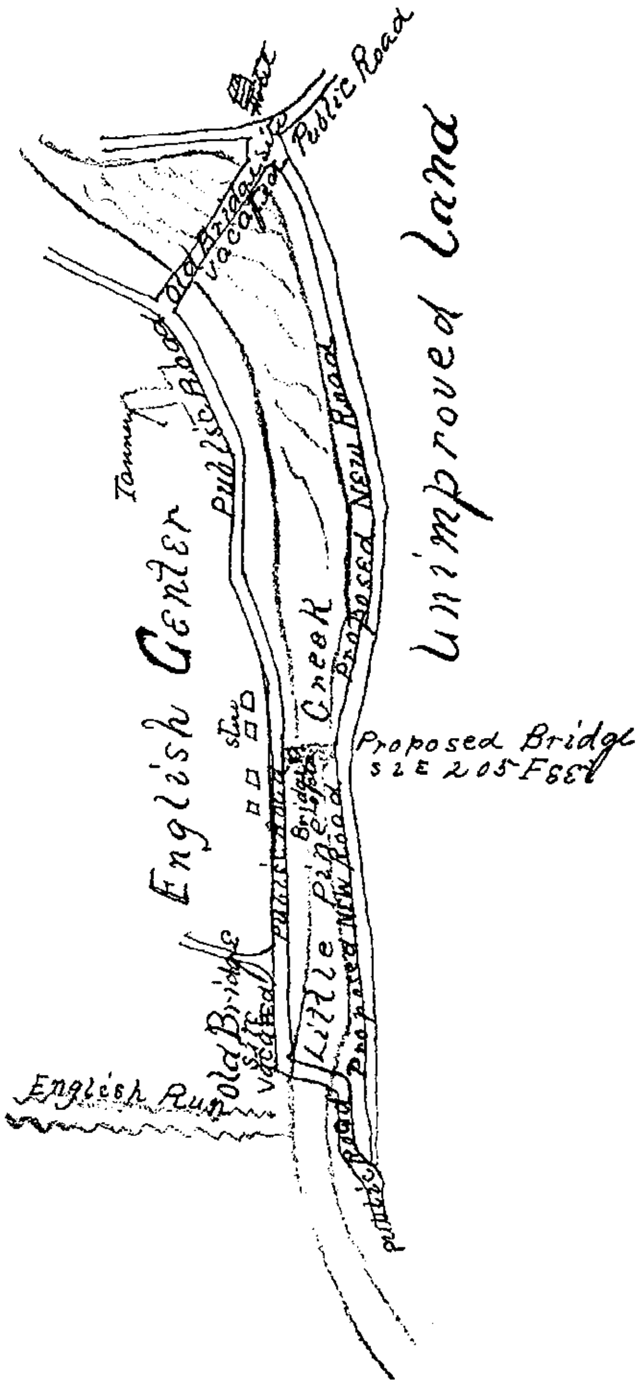
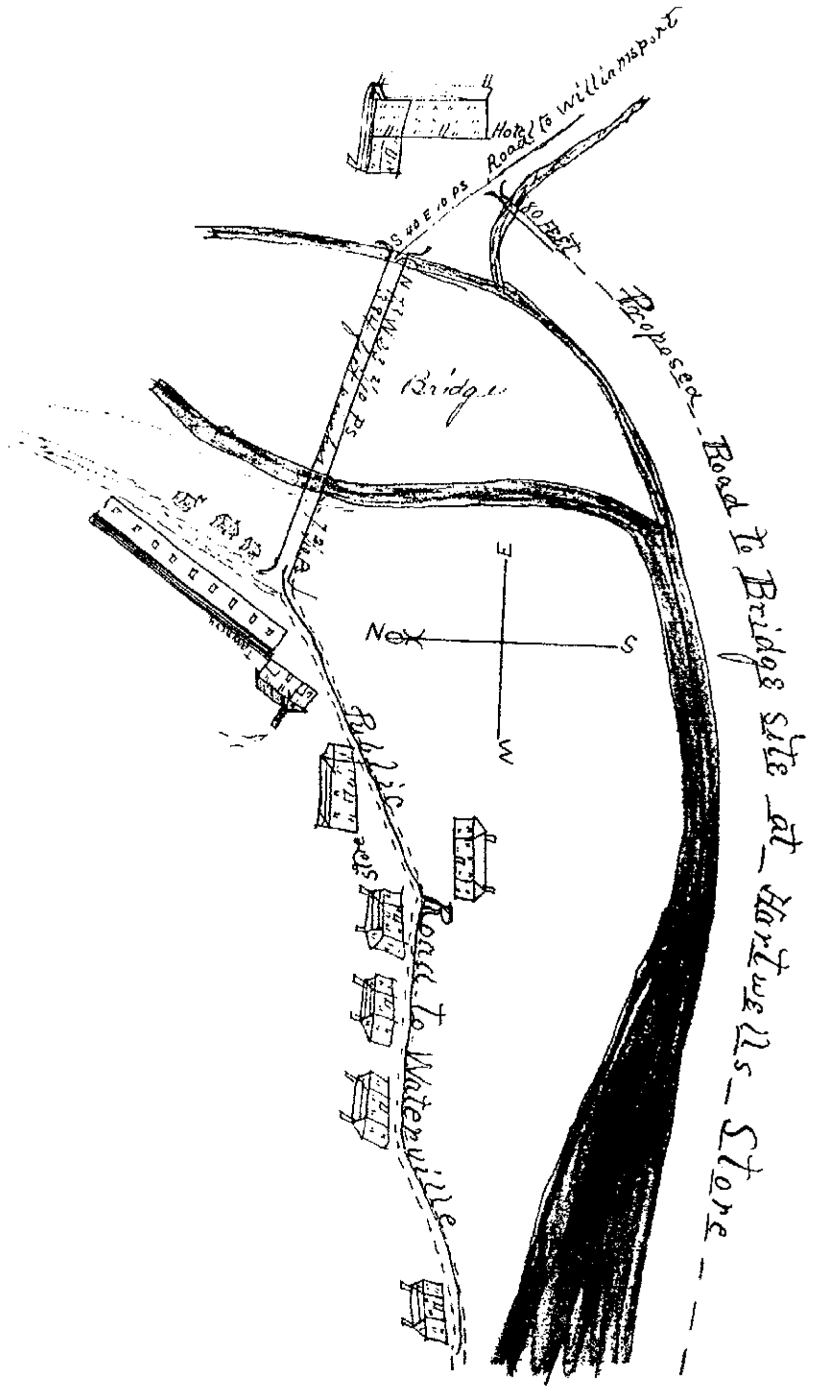


Figure 2 "Map from Report of Reviewers," 29 November 1889, from Lycoming County, *Quarter Session Court Dockets,* No. 20 (September 1889) (Office of Prothonotary, Lycoming County Courthouse, Williamsport, Pennsylvania).



HISTORIC AMERICAN ENGINEERING RECORD

LOWER BRIDGE AT ENGLISH CENTER

This structural study supplements a 12-page historical report written during the summer of 1997.

Abstract:

Although its eye-bar chain and longitudinal deck girder give the appearance of a suspension bridge, the Lower Bridge at English Center, in Lycoming County, Pennsylvania, carries live loads primarily by truss action along its 300-foot span. It is the lone survivor of a pair of bridges erected by Dean and Westbrook in 1891, both containing diagonals crossing each panel between chain and deck, and verticals apparently designed as compression members. As structural analysis and load testing of the Lower Bridge reveal, these members contribute to its truss-like behavior. Two unknowns remain, however: at what point during the erection procedure the diagonals were tightened, and whether any pretension was applied.

Because each end of the deck is detailed to allow horizontal displacement, the English Center bridges' structural form can be described as a "two-hinged inverted trussed arch." The trussing stiffens the flexible chain against moving live loads and wind, a need recognized by designers and users of suspension structures throughout their history. In addition to the deck-stiffening girders common among modern suspension bridges, other stiffening systems include "braced chains," stays, cable trusses, and the inverted trussed arches found at English Center. Structural analysis of several of these alternate systems provides a basis for comparison with the Lower Bridge. Results show that without the trussing between catenary and deck, a much heavier longitudinal girder would be required to provide equivalent vertical stiffness. The inverted arch form, therefore, requires substantially less material than a conventional deck-stiffened suspension bridge for this short span.

Engineers:

Justin M. Spivey; Dr. Thomas E. Boothby, P.E.; Dr. Dario A. Gasparini, P.E.; and Stephen G. Buonopane; August 1998.

Project Description: The Pennsylvania Historic Bridges Recording Project - II was co-sponsored during the summer of 1998 by HABS/HAER under the general direction of E. Blaine Cliver, Chief; the Pennsylvania Department of Transportation, Bureau of Environmental Quality, Wayne W. Kober, Director; and the Pennsylvania Historical and Museum Commission, Brent D. Glass, Executive Director and State Historic Preservation Officer. The field work, measured drawings, historical reports, and photographs were prepared under the direction of Eric DeLony, Chief of HAER.

TABLE OF CONTENTS

1. INTRODUCTION

The Lower Bridge at English Center and its demolished twin Upper Bridge are examples of an uncommon suspension bridge type with eye-bar chains and full trussing between chain and deck.[23] Both were constructed across Little Pine Creek at English Center, Lycoming County, Pennsylvania, by Dean and Westbrook in 1891. The Lower Bridge was placed in the National Register of Historic Places in 1978 and documented by HAER in 1997. In his narrative history of the bridge, Dr. Mark M. Brown indicated that "the unusual structural system has generated a variety of ideas about how the bridge behaves."[24] This report endeavors to provide a conclusive explanation.

Before discussing the English Center bridge, this report gives an overview of suspension bridge forms and the English Center bridge's history, then summarizes previous ideas about its behavior. Descriptions of a linear elastic analytical model and experimental load testing precede a discussion of results from both. Both linear and non-linear models were developed to compare the Lower Bridge at English Center's form to other suspension bridge forms, and results from these additional models are then discussed. Specific observations about the English Center bridge's behavior conclude this report.

The Lower Bridge at English Center, as noted by Dr. Dario A. Gasparini, P.E., in the previous HAER report, can carry loads by either suspension or truss behavior.[25] This study shows that truss action dominates for live loads. Experimental load testing confirms analytical predictions, and also uncovers limitations of the structural analysis. Some questions remain regarding the effect of erection and repair sequences on member prestress, but these are beyond the scope of linear elastic analysis.

2. SUSPENSION AND RELATED BRIDGE FORMS

Suspension bridges have inspired an extensive body of literature ranging from poetry, to histories of individuals and bridges, to technical articles. Perhaps the pre-eminent western technical treatise is Navier's *Rapport a Monsieur Becquey ... et mémoire sur les ponts suspendus,* published in 1823.[26] In 1941, Jakkula compiled an invaluable bibliography of literature up to

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[23] See measured drawings, U.S. Department of the Interior, Historic American Engineering Record (HAER), No. PA-461, "Lower Bridge at English Center," 1997, Prints and Photographs Division, Library of Congress, Washington, D.C.

[24] HAER No. PA-461, "Lower Bridge at English Center," 7.

[25] HAER No. PA-461, "Lower Bridge at English Center," 8.

[26] Claude L. M. H. Navier, *Rapport a Monsieur Becquey, conseiller d'etat, directeur general des ponts et chausees et des mines; et mémoire sur les ponts suspendus* (Paris: Imprimérie Royal, 1823).

that time.[27] A small sample of work on suspension bridges may include Rankine, Melan, Bender, Seguin, Moisseiff, Ammann, Steinman, Pugsley, and, on the origins of the form, Needham.[28] Steinman categorized various suspension bridge forms, discussed advantages and disadvantages of each, and provided illustrations of actual designs.[29] More recent historical perspectives and insights are given by Billington, Peters, Kemp, and Buonopane.[30] Many writings focus on the dominant form: the deck-stiffened suspension bridge. Designers have, however, developed a rich variety of alternate structural forms to stiffen suspension chains or cables. These alternate forms are discussed here to provide perspective on the Lower Bridge at English Center's structural behavior. Qualitative comparisons reveal issues of structural behavior and designers' judgments. Some of these issues are unresolved. In fact, providing adequate stiffness and stability in wind, and modeling geometrically non-linear behavior, remain design challenges today.

The specific forms to be discussed are shown schematically in Fig. 3. The unstiffened cable (Fig. 3a) is discussed first to provide perspective on the nature of the design problems.

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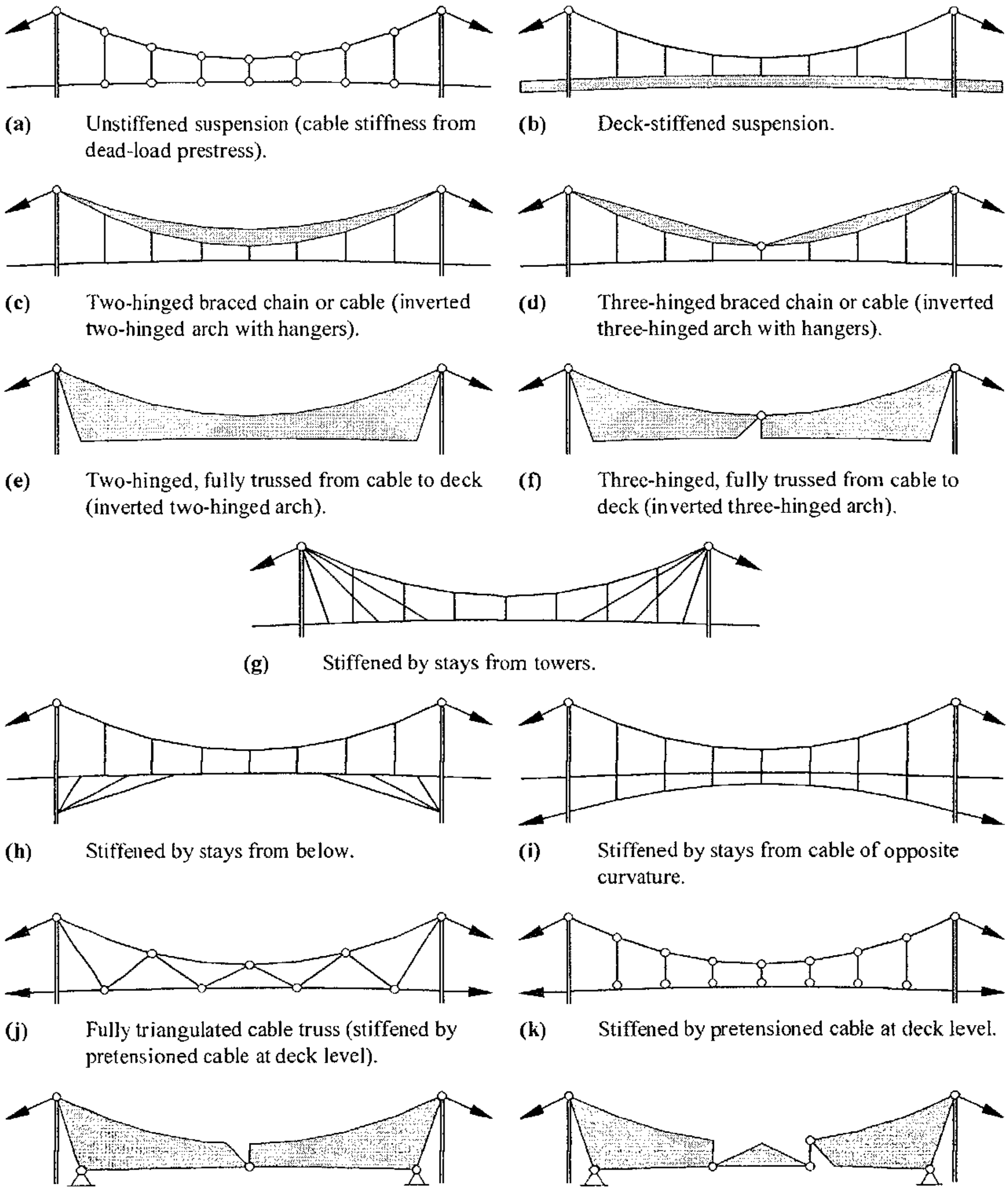
[27] Arne A. Jakkula, "A History of Suspension Bridges in Bibliographical Form," *Bulletin of the Agricultural and Mechanical College of Texas,* 4th series, vol. 12, No. 7 (1 July 1941).

[28] William Rankine, *A Manual of Applied Mechanics,* 5th ed. (London: Charles Griffin and Company, 1869); Josef Melan, "Theorie der eisernen Bogenbrücken und der Hängenbrucken," *Handbuch der Ingenieurwissenschaften,* vol. 2, part 4 (Leipzig: Wilhelm Engelmann, 1888); Charles Bender, "Historical Sketch of the Successive Improvements in Suspension Bridges to the Present Time," *Transactions of the American Society of Civil Engineers* 1 (1872): 28-43; Othmar H. Ammann, "Possibilities of the Modern Suspension Bridge for Moderate Spans," *Engineering News-Record* 90, No. 25 (21 June 1923): 1072-78; Marc Seguin, *Des ponts en fil de fer* (Paris: Bachelier, 1824); Leon Moisseiff, "The Towers, Cables and Stiffening Trusses of the Bridge over the Delaware River between Philadelphia and Camden," *Journal of the Franklin Institute* 200, No. 4 (October 1925): 436-66; David B. Steinman, *A Practical Treatise on Suspension Bridges,* 2nd ed. (New York: John Wiley and Sons, 1929); Alfred G. Pugsley, *The Theory of Suspension Bridges* (London: Edward Arnold, 1968); Joseph Needham, *Science and Civilization in China,* vol. 4, part 3, *Civil engineering, including bridges and canals; nautics* (Cambridge: Cambridge University Press, 1954).

[29] Steinman, *Practical Treatise,* 2nd ed.

[30] David P. Billington, "History and Esthetics in Suspension Bridges," ASCE *Journal of the Structural Division* 103, No. 8 (August 1977): 1655-72; Tom F. Peters, *The Development of Long-Span Bridge Building* (Zurich: Verlag der Fachüreine, 1979); Emory L. Kemp, "Links in a Chain: The Development of Suspension Bridges 1801-1870," *The Structural Engineer,* No. 8 (August 1979): 255-63; Stephen G. Buonopane and David P. Billington, "Theory and History of Suspension Bridge Design From 1823 to 1940," ASCE *Journal of Structural Engineering* 119, No. 3 (March 1993): 954-77.



(a) Unstiffened suspension (cable stiffness from dead-load prestress).

(b) Deck-stiffened suspension.

(c) Two-hinged braced chain or cable (inverted two-hinged arch with hangers).

(d) Three-hinged braced chain or cable (inverted three-hinged arch with hangers).

(e) Two-hinged, fully trussed from cable to deck (inverted two-hinged arch).

(f) Three-hinged, fully trussed from cable to deck (inverted three-hinged arch).

(g) Stiffened by stays from towers.

(h) Stiffened by stays from below.

(i) Stiffened by stays from cable of opposite curvature.

(j) Fully triangulated cable truss (stiffened by pretensioned cable at deck level).

(k) Stiffened by pretensioned cable at deck level.

(l) Cantilever form with no supported span.

(m) Cantilever form with central supported span.

Figure 3 Typology of suspension and related bridge forms. Shading indicates regions of flexural stiffness.

2.1. Unstiffened Suspension Bridges

Fig. 3a shows an unstiffened cahle or chain with vertical hangers supporting a roadway. Many early British suspension hridges, including Samuel Brown's Union Bridge, built in 1820 over the Tweed, were unstiffcned. Navier's 1823 *Rapport a Monsieur Becquey,* written after two inspection tours of British bridges, is the origin of a theoretical basis for the design of unstiffened suspension bridges.[31] Some theoretical results regarding the static behavior of unstiffened cables are illustrated qualitatively in Fig. 4. Fig. 4a shows a vertical live load, *L,* applied to a cable of axial stiffness *EA,* span *S,* sag *H,* and negligible weight relative to *L*. The cable will have no stiffness until a bilinear equilibrium form is reached, with a corresponding vertical displacement, Δ_1. The equilibrium shape is very different from the original shape. Because the equilibrium position is not known *a priori* and because large displacements occur before an equilibrium shape is attained, the cable is said to be geometrically non-linear. If there is a dead load, *D*. uniformly distributed along the horizontal projection of the cable, the cable will have a parabolic equilibrium shape. If a live load, *L,* is applied as shown in Fig. 4b, the cable will have stiffness — meaning that a finite load is needed to cause a displacement — due to axial strain from dead load. The vertical displacement, Δ_2, will be smaller than Δ_1.

[31] Buonopane and Billington, "Theory and History," 955.

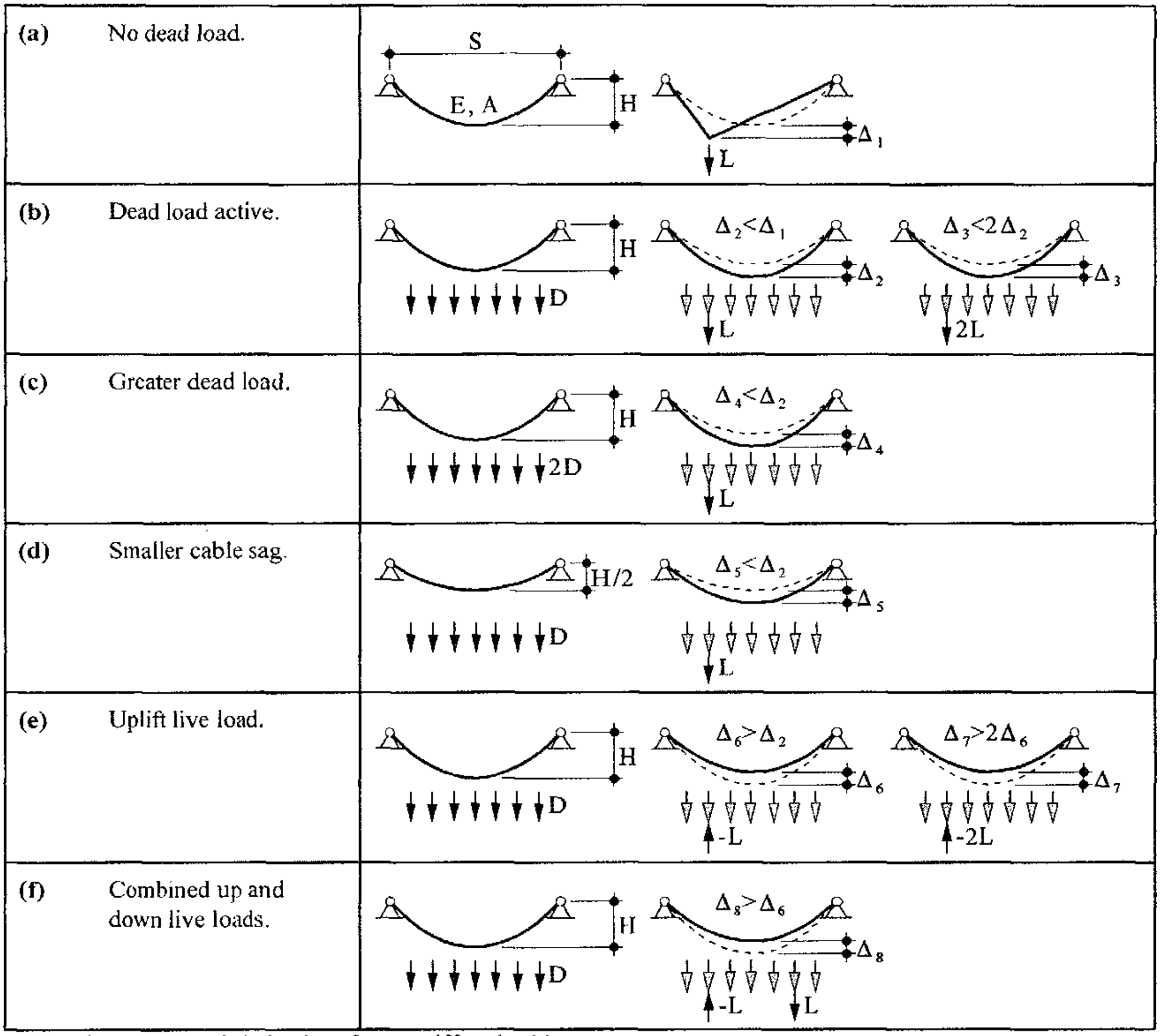


Figure 4 Static behavior of an unstiffened cable.

If the live load is doubled to $2L$, the corresponding displacement, Δ_3, will be smaller than $2\Delta_2$. Because of this, the cable is said to be a hardening system for gravity live loads. If the dead load is doubled as shown in Fig. 4c, and then the live load, L, is applied, the corresponding displacement, Δ_4, will be smaller than Δ_2 because the ratio of live load to dead load, L/D, was decreased. In general, L/D is one of the dimensionless parameters that determines whether the equilibrium shape under combined dead and live loads is significantly different from the parabolic shape under dead load only. If the same dead load, D, and live load, L, are applied to a cable with a smaller sag, say $H/2$ as shown in Fig. 4d, the corresponding live-load vertical displacement, Δ_5, will be smaller than Δ_2 because the sag-to-span ratio, H/S, has decreased. That is, for a given span and *L/D* ratio, the cable with the smaller sag will have a greater vertical stiffness for gravity live loads.

Given an uplift live load, *-L*, applied as shown in Fig. 4e, the corresponding vertical displacement, Δ_6, will be greater than Δ_2; i.e., a vertical stiffness for an uplift load is smaller than a vertical stiffness for a gravity live load. If the uplift load is doubled to $-2L$, the corresponding vertical displacement, Δ_7, will be greater than $2\Delta_6$. Because of this, the cable is said to be a softening system for uplift live loads. If there are two live loads, one an uplift force and the other a downward force as shown in Fig. 4f, the corresponding vertical displacement, Δ_8, will be greater than Δ_6. In summary, the following general observations may be made for unstiffened cables:

1. The sag-to-span ratio, *H/S,* the ratio of live load to dead load, *L/D,* and a nondimensional dead load, *D/EA,* control the static behavior of a suspended cable.
2. The dead load gives a cable some effective stiffness for a vertical live load.
3. For a given span and dead load, an effective cable vertical stiffness for a gravity live load increases as *H/S* and *L/D* decrease.
4. A cable is in general geometrically non-linear, either hardening or softening, depending on the direction of the live load.
5. There is no unique vertical live-load stiffness from a dead load. For any dead load, an effective vertical live-load tangent stiffness depends on the magnitude, direction, and position of the live load.

Although it is not shown quantitatively here, for the antisymmetric live load pattern shown in Fig. 4f, *L/D* ratios as small as 0.05 can cause significant vertical static displacements. Moreover, if a live load varies with a frequency that is close to an effective natural frequency of a cable, static displacements may be significantly amplified.

On the basis of his theoretical results and actual nineteenth-century *L/D* ratios, Navier concluded that unstiffened suspension cables can function without excessive vertical displacements from gravity live loads. Because an effective vertical stiffness for gravity live loads increases as *H/S* decreases, Navier advocated cables with small sag-to-span ratios, from 1/12 to 1/15.[32] Although Navier was definitely aware of wind effects from his studies of British bridges, neither he nor anyone else was able to present an analytical treatment at the time.[33] (Rigorous theoretical and experimental investigation of aerodynamic effects, in fact, did not occur until after the Tacoma Narrows collapse in 1940.) Navier therefore wrote only a

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[32] See Navier, *Rapport a Monsieur Becquey.*

[33] Many early nineteenth-century British unstiffened suspension bridges were damaged or destroyed by wind; see J. K. Finch, "Wind Failures of Suspension Bridges: or Evolution and Decay of the Stiffening Truss," *Engineering News-Record* 126 (13 March 1941): 403.

cautionary statement recommending insight based on experience and observation to prevent wind-induced problems.[34]

Charles Ellet designed and built the Wheeling Suspension Bridge in 1848 as an unstiffened cable with a small sag-to-span ratio, following Navier's principles.[35] The bridge's cables and vertical suspenders were severely damaged, and its wooden flooring destroyed, during a wind storm in 1854.[36] In contrast, James Finley of Fayette County, Pennsylvania, the pioneering designer of suspension bridges in the United States, built bridges with roadways stiffened by trusses beginning in 1801.[37] Such trusses, without doubt, were empirically designed to decrease displacements from moving gravity live loads. They also improved the performance of suspension bridges in wind storms.

2.2. Deck-Stiffened Suspension Bridges

The deck-stiffened suspension bridge, shown schematically in Fig. 3b, has become the dominant form. Following in the tradition of Finley's bridges, later deck-stiffened examples in Pennsylvania include the 388'-long Kellams Bridge at Stalker (1889), the three-span Riegelsville Bridge (1904), and Philadelphia's Benjamin Franklin (1926) and Walt Whitman (1957) bridges, all spanning the Delaware River; Pittsburgh's South Tenth Street Bridge (1933) spanning the Monongahela; and others.[38] A stiff deck is often considered to be a defining characteristic of a suspension bridge. Buonopane and Billington have written a thoughtful analysis of the history of methods used to design stiffening trusses (or girders).[39] There were two principal methods: an earlier, geometrically linear procedure known as the Elastic Theory and a later, geometrically non-linear procedure called the Deflection Theory. The latter was embraced as a significant improvement because it considered an effective vertical cable stiffness from dead load and thus allowed a more economical stiffening truss or girder.

Until 1940, most designers believed that non-uniform gravity live loads controlled the required strength and stiffness of the truss or girder. Therefore only non-uniform gravity live loads were used with both geometrically non-linear and linear *planar* analyses. Antisymmetric wind loads, as shown in Fig. 4f, were not considered critical for design and hence not included in analyses. It was assumed that on long spans, the corresponding great dead load would provide

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[34] See Navier, *Rapport a Monsieur Becquey*, 161.

[35] See HAER No. WV-2 for documentation of the Wheeling Suspension Bridge. Although Ellet's bridge had wooden railings along either side of its deck, he did not consider these as stiffening trusses.

[36] Finch, "Wind Failures," 404-405.

[37] Finch, "Wind Failures," 402.

[38] Appendix A is a list of suspension bridges in Pennsylvania, not all of which are deck-stiffened. See HAER No. PA-470 for documentation of Kellams Bridge, and HAER No. PA-31 for the Riegelsville Bridge.

[39] Buonopane and Billington, "Theory and History."

stability in wind. In his 1929 book, Steinman stated that the depth of stiffening trusses may be "1/90th to 1/150th of the span, for spans up to 3000 feet; and the stiffening trusses may be dispensed with for longer spans. The increasing ratio of dead load to live load reduces the need for extraneous stiffening."[40] This conclusion was based on the results of geometrically non-linear analyses for non-uniform gravity live loads. The effective cable stiffness from dead load is much smaller for antisymmetric live loads, however, as indicated qualitatively in Fig. 4f. The collapse of the Tacoma Narrows Bridge in 1940 showed that deck stiffening may still be necessary for control of antisymmetric displacements from wind, even though it may no longer be necessary (because of a large dead load) for control of vertical displacements from non-uniform gravity live loads. It also showed that torsional stiffness of the deck was an important design parameter and that there was a need to consider aerodynamic behavior.

An important design decision regarding stiffening trusses was whether to use continuous, two-hinged, or three-hinged forms. Steinman discussed the relative advantages of each.[41] The three-hinged form, with a hinge at mid-span, is statically determinate but requires detailing connections that accommodate large relative rotations at mid-span; he therefore advocated the simply supported two-hinged form even though it produces a statically indeterminate suspension structure.

2.3. Braced-Chain Suspension Bridges

Vertical stiffness for gravity live loads may be achieved by stiffening the suspension chain or cable rather than the deck. Steinman referred to such designs as "braced chains."[42] Figs. 3c and 3d schematically show two-hinged and three-hinged braced chains. The origin and development of such designs needs to be studied in depth. Bender stated:

> In the year 1842, the Austrian engineer, Schnirch, ... proposed a new system of suspension bridges, the curves consisting of two pairs of parallel chains, one above the other, and both connected by triangular trussing.... Schnirch's plan was carried out for two tracks in Vienna, in the year 1861, to bridge a canal of the Danube River.[43]

Tyrrell stated: "The Weser suspension bridge, near Hameln, by Wendelstadt (1839) was the first use of triangular bracing between double chain cables."[44] Ammann stated: "The Weser Bridge at Hameln, built in 1836 and still in use, was the first one in which a complete diagonal system

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[40] Steinman, *Practical Treatise,* 2nd ed., 83.

[41] Steinman, *Practical Treatise,* 2nd ed.

[42] Steinman, *Practical Treatise,* 2nd ed., 18ff.

[43] Bender, "Historical Sketch," 41.

[44] Henry G. Tyrrell, *History of Bridge Engineering* (Chicago, 1911), 21.

between a pair of chains was introduced.... In 1860 such a system was used in the railroad bridge across the Danube Canal in Vienna."[45] As an alternative to trussing, A. E. Cowper in 1847 proposed to "construct the chains of boiler plate of considerable depth — say three or four feet, or more — and rivet the whole well together without any movable joints, or separate links.... I propose to call bridges made on this plan, 'Inverted-Arch Bridges.'"[46]

In a series of articles in 1875 and 1876 in *Engineering*, T. Claxton Fidler suggested a particular form of the three-hinged inverted braced arch. Fidler proposed using straight upper chords tangent at their center to the "parabolic curve of the lower chain of the other half-rib."[47] Fidler emphasized that such geometry facilitates the transfer of forces at the center pin.

Several prominent braced-chain bridges were built in the United States. Perhaps the first was Edward Hemberle's Point Bridge, built from 1876 to 1877 across the Monongahela River in Pittsburgh. Gustav Lindenthal also used a braced chain for his Seventh Street Bridge, built in 1884 across the Allegheny River at Pittsburgh.[48] In 1890 Carl Gayler designed the Grand Avenue Bridge in St. Louis as a three-hinged inverted arch with a center span of 400 feet.[49] The bridge was fabricated and built by the King Iron Bridge and Manufacturing Company of Cleveland, with Frank C. Osborn as chief engineer.[50] By far the longest-span braced-chain design was Gustav Lindenthal's unbuilt 1921 proposal for a Hudson River bridge, illustrated in Steinman's book.[51] The span was 3240 feet and the deck 235 feet wide, with two levels accommodating twelve railroad tracks, four bus lanes, and sixteen car lanes. The most notable surviving example of a braced-chain bridge is the Tower Bridge in London, built by Sir Horace Jones in 1895.

Braced-chain suspension forms were conceptualized, analyzed, designed, and designated as "inverted arches." Because a braced-chain is flexurally stiff, it carries moments from non-uniform gravity live loads, especially near the quarter points. These live-load moments, when added to the dead-load effects, should not produce net compressive forces in members of the

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[45] Ammann, "Possibilities," 1074.

[46] A. E. Cowper, "Railway Suspension Bridge," *The Civil Engineer and Architect's Journal* 10, No. 123 (December 1847): 369.

[47] T. Claxton Fidler, "Suspension Bridges and Arches," *Engineering* (London) 19 (30 April 1875): 372-74; "Arched-Ribs and Suspension Bridges, with Their Auxiliaries," *Engineering* (London) 20 (5 November 1875): 351-52, (3 December 1875): 429-30, (24 December 1875): 487-88, and (31 December 1875): 509-510; 21 (28 January 1876): 63-64 and (10 March 1876): 183-84.

[48] Jakkula, "A History of Suspension Bridges," 238.

[49] "Grand Avenue Bridge, St. Louis, Mo.," *Engineering News* 24 (16 June 1891): 8-9.

[50] David A. Simmons, "Bridge Building on a National Scale: The King Iron Bridge and Manufacturing Company," *IA: The Journal of the Society for Industrial Archeology* 15, No. 2 (1989): 35.

[51] Steinman, *Practical Treatise*, 2nd ed., 110-11.

chain, which is *laterally* unbraced. In general, the behavior of braced-chain bridges is very different from that of deck-stiffened suspension bridges. Steinman discussed the relative advantages of braced chains.[52] Such forms were considered to be inherently stiffer and hence were favored for railroad bridges. Braced chains were more compatible with eye-bar construction, which, for longer spans, was not as economical as cable spinning.

The only known extant braced-cable bridges are a multiple-span pedestrian bridge over the Delaware River at Lumberville, Pennsylvania, built by the John A. Roebling's Sons Company in 1947, and a single-span prototype between two buildings at the Roebling works in Trenton, New Jersey.[53] The Lumberville bridge, with five spans totaling 688 feet, contains four and a half two-hinged inverted arches.

The three-hinged braced chain was considered to be statically determinate and therefore had the advantages of easy analysis, no "ambiguity of stresses," and no significant stresses from changes in temperature. The form was perceived to be linear in its behavior, did not require stiffening of the deck, and was therefore an "easier" form for engineers to accept and design. Engineers did not need to consider the difficult concept of cable stiffness from dead load, nor did they need to make the difficult decision on the amount of deck stiffness to provide. However, the relative structural performance of braced-chain and deck-stiffened suspension forms remains an open issue to this day. Specifically at question is whether the linear live-load stiffness provided by an inverted arch is greater than the geometrically non-linear live-load stiffness resulting from small changes in the geometry of an inverted arch carrying a large dead load. Unlike in the deck-stiffened form, the flexural stiffness of the braced chain inhibits the development of this geometrically non-linear stiffness. Whether this is structurally desirable is an open issue.

Braced chains do retain one source of non-linear behavior. If the deck is unstiffened and cable hangers are used, there is a possibility of the hangers becoming slack from uplift forces from wind. By fully trussing the suspension form between the chain (or cable) and the deck, such non-linear behavior is precluded.

2.4. Fully Trussed Suspension Bridges

Figs. 3e and 3f conceptually show fully trussed, two- and three-hinged inverted arch forms. As with braced chains, the origin and development of fully trussed inverted arch forms needs to be studied in depth. Briseghella recently documented a fully trussed suspension bridge

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[52] Steinman, *Practical Treatise,* 2nd ed., 108-110.

[53] "Suspension Footbridge," *Engineering News-Record* 140 (26 February 1948): 9. Clifford W. Zink and Dorothy W. Hartman briefly discussed these bridges in *Spanning the Industrial Age: The John A. Roebling's Sons Company* (Trenton, New Jersey: Trenton Roebling Community Development Corporation, 1992), 151, 165.

designed by Anton Galateo and erected in 1828 at Padova, Italy.[54] Fidler and Steinman cited the Lambeth Bridge in London, designed by Peter Barlow, as a cable suspension bridge with cross-bracing between the cable and the deck.[55] The bridge was built in 1863 and had a span of 280 feet. Cowper, in 1867, modified his 1847 design with a proposal for a fully trussed inverted arch.[56] Fidler and Steinman cited a pedestrian bridge over the Main River, between Frankfurt and Sachsenhausen, as an example of an inverted three-hinged arch. The bridge was designed by Schnirch, had a span of 262 feet, and was built in 1869.[57]

Like braced chains, fully trussed suspension forms were conceptualized and designed as inverted arches. Yet their behavior is different from that of braced chains with hangers because axial forces in the deck contribute to the arch action. Steinman noted that: "When the braced-chain (or braced-cable) system is used, the web members should preferably not be connected until full dead load and half live load are on the structure at mean temperature."[58] Therefore, if such an erection procedure is followed, such bridges behave as pure suspension forms for carrying dead loads (and a uniform half live load) and as inverted arches for non-uniform live load.

The Lower Bridge at English Center, Pennsylvania, is fully trussed between its eye-bar chain and deck. It has horizontally slotted girder-to-tower connections at both ends and is therefore of the inverted two-hinged trussed arch form. Its behavior is more complex because of the use of counters, whose action depends on when during the erection sequence they were tensioned and on the magnitude of the pretension forces. For the small 300-foot span, it is not likely that the geometrically non-linear stiffness of the chain due to the dead load provides a significant contribution to the bridge's live-load stiffness. This observation may not be correct for the modem suspension bridges designed by Freeman, Fox, and Partners over the Severn, the Humber, and the Bosporus, which have inclined hangers and thus appear as fully trussed systems. (The second Bosporus bridge, for which Dr. William Brown was the design engineer, has vertical hangers.) The large scale may make the geometrically non-linear cable stiffness from dead load dominant.

2.5. Suspension Bridges Stiffened by Cable Stays

A third general way to stiffen a suspension cable is by adding stays. Figs. 3g, 3h, and 3i show three general arrangements. In reality, a bewildering variety of alternate stay arrangements

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[54] Lucia Briseghella, "Anton Claudio Galateo e I ponti Sospesi a Cavi Dell'inizio del XIX secolo," Tesi de Laurea, Instituto Universitario di Architettura di Venezia, 1996.

[55] Fidler, "Suspension Bridges and Arches"; Steinman, *Practical Treatise,* 2nd ed., 80.

[56] A. E. Cowper, "Cowper's Inverted-Arch Suspension Bridge," *Engineering* 3 (22 March 1867): 277.

[57] Fidler, "Suspension Bridges and Arches," 373; Steinman, *Practical Treatise,* 2nd ed., 80.

[58] Steinman, *Practical Treatise,* 2nd ed., 108.

have been tried, often in combination with stiffening trusses. Bender stated that "the English engineers were the first who proposed and applied stays for suspension bridges, in connection with cables, as well as without them," and "the first of these bridges was built by Richard Lees, at Galashiel, across the River Gala, in the year 1816, of a span of one hundred and twelve feet." Bender further stated that "a great many of the stay-bridges broke down, and after many sad experiences the use of stays has been entirely rejected by English engineers."[59] Ammann also stated that "inclined stays, first introduced in England and widely used there in the early part of the nineteenth century, proved particularly fateful. A number of suspension bridges with such stays failed, on account of insufficient resistance to wind pressure and this led to the complete abandonment of that type in England."[60] In recognition of Roebling's achievements, Ammann added:

> In the second half of the 19th century inclined stays were revived in America; in connection with the stiffening truss and efficient lateral bracing, introduced meanwhile, they proved more effective, although they led to occasional trouble an account of uncertain distribution of the load.[61]

Steinman devoted only a six-line paragraph in his book to the use of stays:

> Another method of stiffening the suspension bridge is by the introduction of diagonal stays between the tower and the roadway. These, however, have the disadvantages of making the stress-action uncertain, and of becoming either overstressed or inoperative under changes of temperature; moreover, they introduce unbalanced stresses in the towers.[62]

Stays generally increase the degree of static indeterminacy of a suspension bridge. It is very difficult to decide on the appropriate level of pretension in stays and to achieve the desired pretension during construction. If stays go slack, the bridge behavior becomes non-linear.

Stays from above (Fig. 3g), if properly pretensioned, can effectively stiffen a suspension bridge and inhibit the kind of motion in which the displaced deck resembles one full sinusoidal cycle. A stay's effectiveness decreases with increasing distance from the tower as the angle between the stay and the deck decreases. Stays from above decrease the dead-load tension in the suspension cable and thus decrease its effective vertical stiffness. If there is a horizontal slip-joint at mid-span, stays induce compression in the deck. Stays from above can cause bending in the tower if they are not continuous over the tower saddle. Stays and hangers share in carrying the live load, in proportion to their stiffnesses.

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[59] Bender, "Historical Sketch," 31.

[60] Ammann, "Possibilities," 1073.

[61] Ammann, "Possibilities," 1073.

[62] Steinman, *Practical Treatise*, 2nd ed., 76.

Stays from below (Fig. 3h), again if properly pretensioned, can effectively stiffen a suspension bridge. They also increase the tension in the suspension cable and thus increase its effective vertical stiffness. As with stays from above, properly pretensioned stays from below will also share in carrying live load. Of course, stays from below may not be practical because they obstruct clearances below a bridge.

Stays from a cable with opposite curvature (Fig. 3i) are, of course, placed all along a span. Their effectiveness does not decrease with distance from a tower. They increase the tension in the suspension cable and thus increase its effective vertical stiffness. If appropriately pretensioned, they share in carrying the live load. If vertical, they do not induce an axial force in the deck. The effectiveness of such a stay configuration in inhibiting the kind of motion in which the displaced deck resembles one full sinusoidal cycle is uncertain because the stays are attached to the lower cable that itself can displace in such a mode.

The use of stays in combination with stiffening trusses was the trademark of John A. Roebling and his son Washington. A variety of stay arrangements were used on the Niagara Gorge railroad bridge, the Covington-Cincinnati (now the Roebling) Bridge, and on their masterpiece, the Brooklyn Bridge.[63] An illustration of the elder Roebling's Smithfield Street Bridge in Pittsburgh shows diagonal stays from above, as on the Brooklyn Bridge.[64] Steinman, in his article on the rehabilitation of the Brooklyn Bridge, pointed out that Roebling used slip joints at the middle of both side spans and in the main span; the stays therefore cause significant axial compressive force in the deck. Steinman also stated that "in the region of the diagonal stays, since the stays partially relieve the vertical suspenders in carrying the dead and live loads of the bridge, the curves of the main cables are flatter than where the loads are carried solely by vertical suspenders."[65]

Stays from a cable of opposite curvature, as shown in Fig. 3i, seem to have first been used by Marc Brunel in 1823. Brunel fabricated and assembled such bridges in Britain, then shipped them to the Island of Bourbon (now Reunion, near Madagascar) in the Indian Ocean. The bridges are illustrated in Navier's 1823 *Rapport a Monsieur Becquey,* which shows a single span and also a two-span version with a central tower. Stays from a cable of opposite curvature have been used often for small pedestrian bridges. One example is the Kaibab Trail pedestrian (and mule) suspension bridge spanning the Colorado River at the bottom of the Grand Canyon,

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[63] See HAER No. KY-20 for documentation of the Covington-Cincinnati Bridge, and HAER No. NY-18 for the Brooklyn Bridge.

[64] John A. Roebling, "Views of the New Wire Suspension Bridge of eight spans over the Monongahela River at Pittsburgh Constructed by John A. Roebling C.E. in 1845 to 1846" (Roebling Collection, Rensselaer Polytechnic Institute, Troy, New York), reprinted in Zink and Hartman, *Spanning the Industrial Age,* Fig. 19.

[65] David B. Steinman, "The Reconstruction of the Brooklyn Bridge," *Columbia Engineering Quarterly* (November 1952): 6.

designed by Ward P. Webber in 1928.[66] The most prominent example in the U.S. is the Royal Gorge bridge spanning the Arkansas River in Colorado, designed by George E. Cole in 1929.[67]

2.6. Subsequent Developments in Suspension Bridges

The stiffening systems discussed thus far, with their associated design methods, enabled designers to create incomparable structures such as the Brooklyn, George Washington, and Golden Gate suspension bridges. With hindsight, it is clear that the actions of wind remained poorly understood and poorly modeled in the design process. The crisis caused by the collapse of the Tacoma Narrows bridge in 1940 was salutary for long-span bridge design engineers. It opened the profession to concepts of aeroelastic vibrations and dynamic stability.[68] It fostered the development of wind-tunnel technology for assessing the performance of suspension bridges in wind and led to the development of mathematical models for predicting wind velocities that cause dynamic instability of bridge decks. Moreover, in the early 1940s, concepts for improved cable stiffening systems were proposed. The staff of Modjeski and Masters, in a 1941 *Engineering News-Record* article, proposed the use of inclined hangers and tested the concept on a small (11'-6" long) model.[69] The staff suggested that inclined hangers would produce stiffer and more highly damped bridges. The concept was used by Freeman, Fox, and Partners on the Severn and Humber bridges, although there is still a need for an objective evaluation of the relative performance of inclined hangers.

Before and after World War II, engineers at John A. Roebling's Sons Company performed research on improved stiffening systems. They constructed innovative pedestrian bridges between two of their buildings in Trenton and over the Delaware River at Lumberville. Their developments culminated with the design for El Puento del Litoral (Coastal Bridge) at San Marcos, El Salvador.[70] The stiffening system used on the San Marcos bridge is shown schematically in Fig. 3j. A cable with a small curvature opposite to that of the main suspension cable is placed at the level of the deck and pretensioned longitudinally. Pretensioning of the lower cable against a system of diagonals and the main suspension cable produces a "pretensioned cable truss." The development of the design is described in a series of articles in

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[66] See HAER No. AZ-1 for documentation of the Kaibab Trail Bridge.

[67] See Jakkula, "A History of Suspension Bridges," 310, for references.

[68] Friedrich Bleich, *The Mathematical Theory of Vibration in Suspension Bridges* (Washington, D.C.: U.S. Department of Commerce, Bureau of Public Roads, 1950).

[69] Staff of Modjeski and Masters, "Suspension Bridges and Wind Resistance," *Engineering News-Record* 127 (23 October 1941): 565-68.

[70] Zink and Hartman, *Spanning The Industrial Age,* 151.

Civil Engineering and *Engineering News-Record*.[71] Charles Sunderland, Blair Birdsall, and Norman Sollenberger collaborated on the design of the bridge, which was constructed in 1952. Birdsall stated that "Mathews and Kenan are now employing it for many of their pipeline suspension bridges," but it appears that the concept was not used again for a long-span suspension bridge. The San Marcos design is innovative. Its system, as well as the similar system shown in Fig. 3k, seems to merit further study. Unfortunately, according to Professor Carlos Rene Perez of the University of El Salvador, the San Marcos Bridge was destroyed by guerrillas in 1982.[72] Its performance over thirty years needs to be documented.

Many other suspension forms, different from those shown in Fig. 3, were also used by early designers. Dredge proposed a tapered-chain system that was widely discussed.[73] French designers used the Gisclard system, with cables descending from the top of each tower in lieu of a catenary, as well as combinations of cable-stayed and suspension forms.[74] These designs are not discussed here because additional studies are needed. Instead, two cantilever forms are shown schematically in Figs. 3l and 3m. Cantilever trusses were sometimes designed to have the aesthetic appearance of a suspension structure. A cantilever form is characterized by the fact that the top chord is in tension and the bottom chord is in compression with axial forces increasing from the mid-span to the supports. A cantilever truss generally has a suspended span as shown in Fig. 3m. One example is the Northampton Street Bridge in Easton, Pennsylvania, where the suspended span is disguised by non-structural members that complete the top chord's parabolic profile.[75] In a few designs, such as New York City's Queensboro Bridge, the suspended span is omitted entirely, as shown in Fig. 3l.[76]

In summary, in the context of the forms discussed above, the bridge at English Center is an example of an inverted fully trussed two-hinged arch, shown schematically in Fig. 3e. Analyses presented here show that its static behavior under live load is different from that of conventional deck-stiffened suspension bridges.

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[71] Blair Birdsall, "A Prophetic Design in an Out-of-the-way Place," *Civil Engineering* 24, No. 9 (September 1954): 574-75; "Cable-stiffened Suspension Bridge Updated," *Engineering News-Record* 150, No. 21 (21 May 1953): 32-39; and N. J. Sollenberger, "'Cable Truss' Design Greatly Increases Stiffness," *Civil Engineering* 24, No. 9 (September 1954): 576-79.

[72] Carlos Rene Perez, to Dario A. Gasparini, 23 July 1998.

[73] For a series of articles on Dredge's design, see Jakkula, "A History of Suspension Bridges," 444-48.

[74] See Ammann, "Possibilities," 1075, or Walter Podlony, Jr., and John F. Fleming, "Historical Development of Cable-Stayed Bridges," ASCE *Journal of the Structural Division* 98, No. 9 (September 1972): 2079.

[75] Mansfield Merriman and Henry S. Jacoby, *A Text-Book on Roofs and Bridges*, Part 4, *Higher Structures* (New York: John Wiley and Sons, 1907), 109. See HAER No. PA-502 for documentation of the Northampton Street Bridge.

[76] See HAER No. NY-19 for documentation of the Queensboro Bridge.

3. LOWER BRIDGE AT ENGLISH CENTER

The Lower Bridge at English Center was built in 1891 by Dean and Westbrook, following a flood which destroyed a previous structure on the site.[77] The present bridge was documented by HAER during the summer of 1997, with a narrative history by Dr. Mark M. Brown. His description of the bridge will be summarized here, but the reader is referred to his report for historical details.[78] The bridge has a 300'-0" clear span across Little Pine Creek, with twelve panels (25'-0" each) between towers. The original members are presumably steel; a steel floor system of deck beams, stringers, and open-grid deck was added in 1953.[79] On either side of the deck, a cambered 24"-deep built-up girder is suspended from a chain of pin-connected eye-bars. The upstream girder has been bent by the impact of flood-borne debris, resulting in a large deformation from the vertical plane.[80] The chains pass over the tops of the towers, and are anchored into concrete deadmen 87'-3" from the north tower and 88'-7" from the south.

Laced vertical members, evidently designed for some compressive load, connect the eye-bar pins to the longitudinal girder. The verticals at U1-L1 and U11-L11 are square boxes; the remainder have an H-shaped cross-section. Looped eye-bar diagonals range in diameter from 3/4" to 1-1/4", increasing toward mid-span. The diagonals can be tightened by adjusting bolts on the underside of bent plates which connect them to the bottom flange of the longitudinal girder. The outermost diagonals, U0-L1 and U12-L11, have obviously been replaced at least once; broken-off stubs of the original diagonals were found in the pin connection at the top of each tower. A 1968 inspection drawing indicates that these were originally 3/4" diameter instead of the present 1".[81] The larger retrofit diagonals, which are welded to plates that fit over the pin nuts, have turnbuckles for tightening after installation. A 1994 inspection of the bridge found it to be in "overall serious condition," with several components of the deck and diagonal U11-L10 of the west truss exhibiting "significant section losses."[82]

No records were found to explain the English Center bridge's erection sequence, or to indicate whether it was consciously prestressed. Prestressing permits slender members to carry some compressive force (in effect, reduced tension) without buckling. This phenomenon was certainly known to nineteenth-century engineers. In fact, prestressing is necessary for the

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[77] HAER No. PA-461, "Lower Bridge at English Center," 3-5.

[78] HAER No. PA-461, "Lower Bridge at English Center."

[79] Previous inspections of the bridge noted that its material was steel; see Harry J. Engel, "300' Span Steel Suspension Bridge Over Little Pine Creek — Inspection Drawing," 24 June 1968 (PennDOT Maintenance District 3-2, Montoursville, Pennsylvania); or A. G. Lichtenstein and Associates, "BMS No. 41-4001-0270-0000, Final Bridge Inspection Report," November 1994 (PennDOT Maintenance District 3-2, Montoursville, Pennsylvania), 2.

[80] This occurred prior to 1968; see Engel, "Inspection Drawing."

[81] Engel, "Inspection Drawing."

[82] Lichtenstein, "BMS No. 41-4001-0270-0000," 2.

stability of Howe trusses, where verticals were tightened to lock diagonals into joint blocks; or Pratt trusses, where diagonals were tightened to lock in verticals.[83] Although the English Center bridge's behavior depends on whether its diagonals were pretensioned, prestressing is not necessary for the bridge's stability.

Because Little Pine Creek is fairly shallow, contractors could have used falsework to support the structure during erection. If falsework was used, it may have been removed before the diagonals were tightened. If at this stage the diagonals were only "snug-tightened," they would not carry any of the dead load. The bridge therefore would have behaved as a conventional deck-stiffened suspension bridge (no diagonals) under dead load, with the suspension chain carrying the full weight. If, however, the diagonals were snug-tightened with falsework still supporting the bridge, the bridge's weight would create an initial tension in the diagonals. In this case, the bridge would have behaved as a two-hinged inverted trussed arch under dead load. Alternately, the diagonals may have been snug-tightened after a uniform live load (a fraction of the design live load) was placed on the bridge, perhaps by using local materials such as timber, stone, or soil. Because the bridge is statically indeterminate, it is also possible that the diagonals were tightened to significant pretensions.

In summary, the original erection sequence and tightening procedure, as well as subsequent flood damage, repairs, and retrofits, all affect the member forces that exist prior to the application of any live load. These initial member forces are unknown for the Lower Bridge at English Center. It is therefore not possible to predict which, if any, diagonals will go slack for any particular live load. In fact, the problem of initial tension in statically indeterminate structures remains a difficulty in structural engineering today.

Brown's report concludes with speculation by civil engineers about the bridge's behavior. Victor Darnell supposed that the Lower Bridge might be a double cantilever. Others concentrated on its suspension behavior. Consulting engineer Jackson Durkee, formerly of Modjeski and Masters, suggested that the cross-bracing was superfluous.[84] Structural analysis and experimental load-testing, however, confirm that these members are essential to the truss action that dominates in carrying live loads.

4. STATIC BEHAVIOR OF THE LOWER BRIDGE AT ENGLISH CENTER

The English Center bridge's structural behavior was predicted by computer analysis of linear elastic models, and confirmed by full-scale load testing. Section 4.1 explains the modeling process, Section 4.2 describes the load testing procedure, and the results of these two methods are compared in Section 4.3.

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[83] Dario A. Gasparini and David Simmons, "American Truss Bridge Connections in the 19th Century, I: 1829-1850," ASCE *Journal of Performance of Constructed Facilities* 11, No. 3 (August 1997): 119-27.

[84] HAER No. PA-461, "Lower Bridge at English Center," 7-8.

4.1. Linear Analysis Models

A two-dimensional, linear elastic frame model of the Lower Bridge at English Center was defined using a commercially available computer structural analysis program.[85] Detailed calculations, which can be found in the field note material for this report, are summarized in Appendix B. Dimensions and member cross-sections were obtained from the 1997 HAER drawings, a 1968 inspection report, and a 1994 study by A. G. Lichtenstein and Associates.[86] Fortunately, no conflicts arose among sources. This model, hereafter called Model A, was analyzed for dead load and concentrated live loads applied to each lower chord at the panel point. Fig. 5 shows the bridge's upstream (east) elevation, so panel-point numbering runs in the opposite direction from that in HAER measured drawings, which show the downstream (west) elevation.

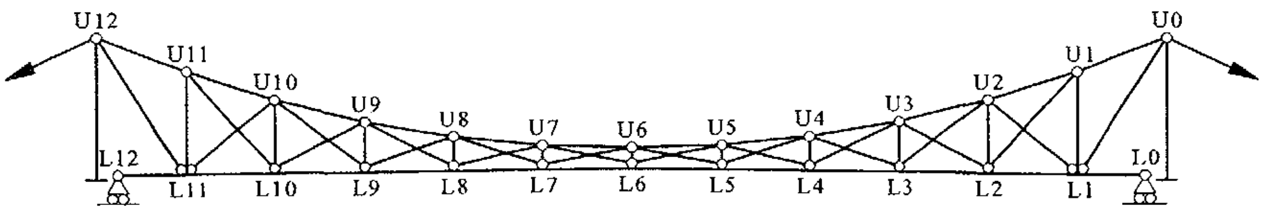

Figure 5 Model A — representing actual geometry of Lower Bridge at English Center (east truss). Panel-point numbering runs from right to left (north to south) to remain consistent with HAER drawings.

Fig. 6 shows the basic types of members: towers, upper chord, lower chord, H-shaped verticals, box-shaped verticals, and four different diagonals. The tapered towers were modeled by discretizing them into three segments, divided at bracing points (see Fig. 6). Properties for each segment were calculated from the plates and angles, using the segment's average depth to determine bending properties. Table B-1 summarizes axial and bending properties calculated for each type of member.

[85] STAAD-III Release 22 for Windows, from Research Engineers, Berkeley, California. The authors are grateful to the PennDOT Bureau of Design for providing a computer workstation on which this software had been installed.

[86] Engel, "Inspection Drawing"; Lichtenstein, "Final Bridge Inspection Report."

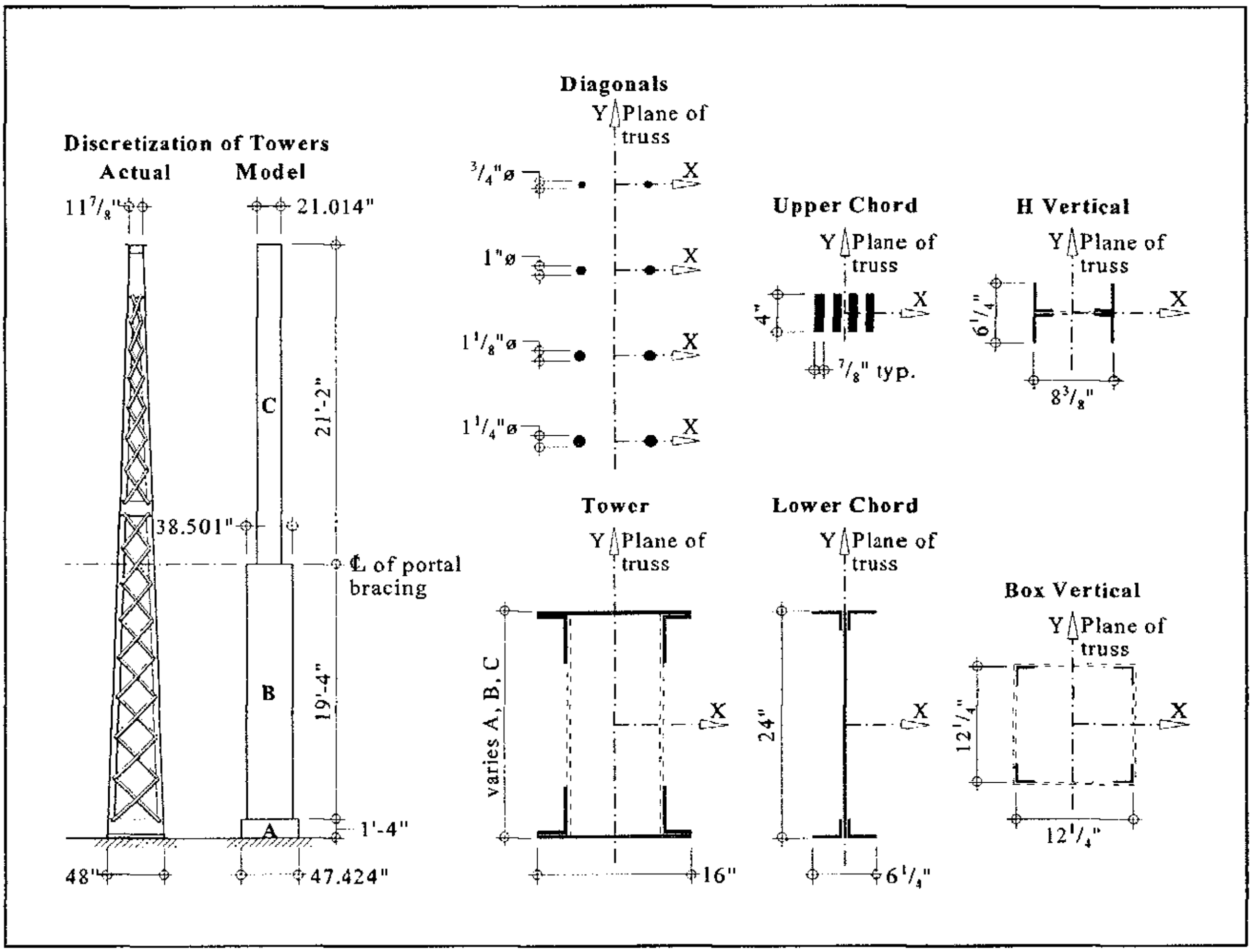

Figure 6 Schematic of member sections used in analytical models.

The structure dead load was calculated from the member properties, assuming a unit weight of 490 pounds per cubic foot (pcf) for steel, and distributing half of each member's weight to the panel points at its ends. Unit weights of floor beams, stringers, open-grid deck, and lateral bracing were also calculated (see Table B-2), and half the deck's weight was distributed to each truss. The total panel-point dead loads are listed in Table B-3.

With the geometry defined and loading determined, the next step was deciding upon member connections and support conditions. For the most part, connections were self-evident. The entire eye-bar chain is pin-connected, and was modeled as such.[87] The H-shaped verticals (at panel points L2 through L10), connected only by a 1/4" plate at their base, seem to have been detailed for a minimum of bending resistance in the truss plane, and were therefore modeled as pin-connected. However, the box-shaped verticals (at panel points L1 and L11) are attached by

[87] While the idealized pinned connections used in the model are frictionless and rotate freely, real pinned connections experience friction and may periodically seize because of corrosion or pin wear.

two 1/4" plates 12" apart, a connection which provides some bending resistance. The tower bases, firmly bolted to the abutment and filled with concrete to prevent their holding water, are certainly a fixed-type support condition. The connection most difficult to model is where the longitudinal girder connects to each tower (panel points L0 and L12). With bolts in horizontally oriented slots, the girder ends were clearly designed to resist vertical displacement while allowing some degree of horizontal displacement and rotation. The girder ends were therefore modeled as a roller-type connection in Model A. Viewing the real connection's behavior under load, however, one can see the bolts displacing not only horizontally, but also vertically, in their slots (confirmed by wear patterns in the slots) and periodically seizing because of rust and other obstructions. Such behavior is non-linear, if not random. An otherwise identical model with pinned-end girders, Model A-P, was constructed to simulate a fully seized state and bracket the real structure's behavior.

Because Model A considers only the plane of the suspension truss, with a lower chord stiffness calculated from the 24"-deep stiffening girder alone, it underestimates the floor system's contribution to bending stiffness. In the actual English Center bridge, the current floor system consists not only of stiffening girders, but also floor beams, stringers, and an open-grid steel deck, for a total depth of 48". The effective cross-sectional area and bending resistance of this system are certainly greater than those of the stiffening girders alone. Although determining an overall stiffness for the floor system is beyond the scope of this report, Model A-D was created to assess the sensitivity of the bridge's structural behavior to the floor's bending stiffness. Results from analysis of this model, identical in geometry to Model A except for a lower chord with twice the cross-sectional area and ten times the in-plane moment of inertia, showed that the structure was relatively insensitive to an order-of-magnitude difference in lower chord stiffness.

A fourth model was created to explore the effect of the slender diagonals buckling under compressive load. Although the actual English Center bridge's diagonals might have significant initial tensions that allow them to carry greater compressive forces without buckling, the magnitudes of these pretensions are not known. Even if they were known, a non-linear model would be required to fully characterize the redistribution of forces as members buckle, a process which depends on the location and magnitude of the applied load. Such a model is beyond the scope of this report. Linear elastic analysis of a model with diagonals incapable of carrying compressive forces, however, gives force results for the extreme case of zero prestress in all diagonals. This is accomplished by removing those diagonals found to carry compressive forces from Model A, then performing a second analysis on the modified structure. Model A-Z takes advantage of the structural analysis program's tension-only members, using them with zero prestress. The program automatically removes tension-only members under compression and reiterates the analysis. Influence lines for unit loads applied to Model A-Z cannot be scaled or superimposed, however, because the force results depend on the location and magnitude of the load.

If falsework was removed from the English Center bridge before tightening its diagonals and counters, those members would not participate in carrying the dead load. Model B, shown in Fig. 7, resembles a conventional deck-stiffened suspension bridge, and was created by deleting the diagonals and counters from Model A. Also, the box-shaped verticals were pinned at L1 and L11, instead of having moment-resisting connections. In this configuration, all verticals act as hangers, carrying only tensile forces for downward loads. Because the hangers act only in the vertical direction, one end of the girder must be pinned to guarantee longitudinal stability of the deck. Large displacements result when the model is loaded — since the 24"-deep lower chord provides only light stiffening — indicating that a geometrically non-linear analysis might provide more accurate results (see Section 5.2).

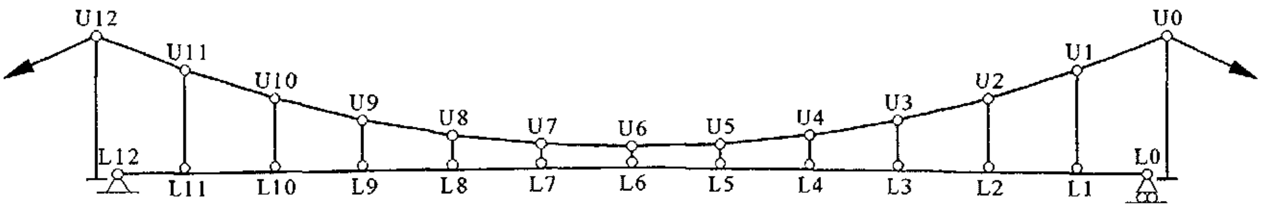

Figure 7 Model B — diagonal members deleted from Model A.

4.2. Experimental Load Testing

The Lower Bridge at English Center was instrumented with strain-measuring devices and load tested on 3 June 1998. Strain (or displacement per unit length) can be measured from a change in length over an original gage length when an external load is applied. Stress and strain are related by a material property called the modulus of elasticity, typically denoted *E*. Assuming that the English Center bridge is made of steel, and that the material remains in the linear elastic range, *E* has a constant value of 29,000 kilopounds per square inch (ksi). Given the member's cross-sectional area, *A*, and moment of inertia, *I*, axial force and bending moment can also be calculated. Three types of strain-measuring devices were attached to selected members of the structure's upstream (east) truss, as shown in Fig. 8. These devices, and the data acquisition system used, are fully described in Appendix C.

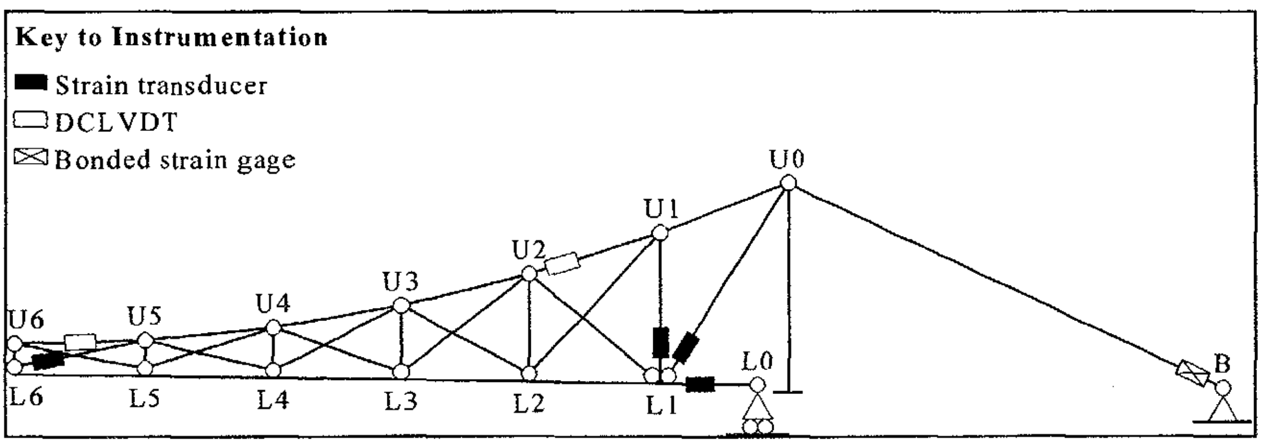

Figure 8 Locations of strain-measuring devices on upstream (east) truss of Lower Bridge at English Center.

The structure was tested using an empty dump truck, provided by PennDOT Maintenance District 3-2, with an axle spacing of 13.6 feet. The truck had a total weight of 18.6 kilopounds (kips), 10.1 kips on the front axle and 8.5 on the rear, measured by PennDOT scales in the field. The test series described in this report was run with the truck centered on the bridge, although subsequent series had the truck as close to the edge on the instrumented east side as possible. Each truck crossing was videotaped to provide a time reference so that truck location could be correlated with the time index on the data acquisition system. The electronic signal from the strain-measuring devices was recorded, and later converted to strain units. The strain records, correlated with truck position, were used to construct influence lines, which display the axial force in a member as a function of live load position. Fig. 9 shows one example of an influence line. In this and subsequent figures, positive axial force values indicate tension while negative values indicate compression.

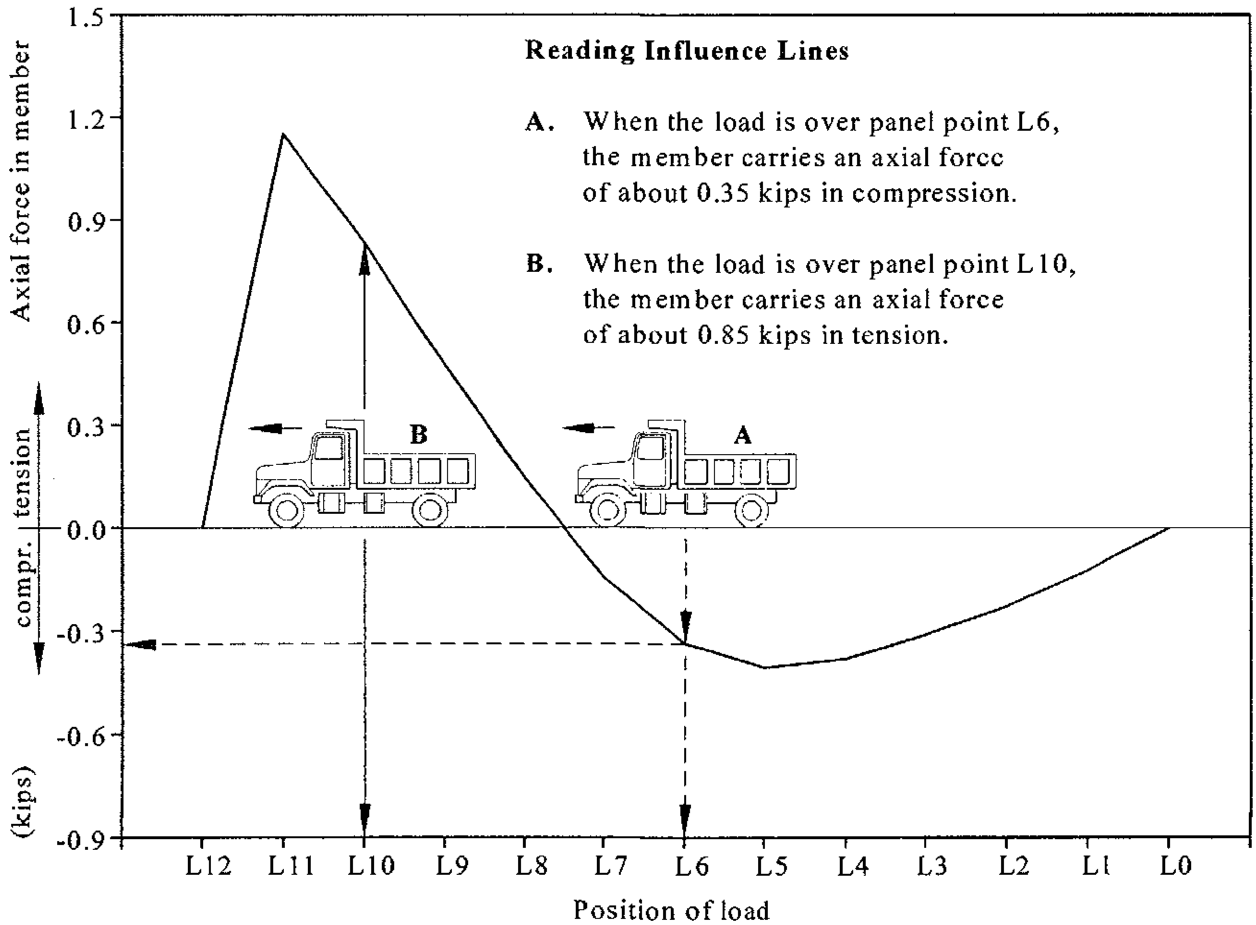

Figure 9 Example of influence line, showing the axial force in a member as a function of load position. Note that the truck's leftward movement is consistent with panel-point numbering in previous and subsequent figures.

The strain data were reduced to axial forces by multiplying by the modulus of elasticity and cross-sectional area of each member, then dividing by 9.3 kips, half of the truck's total weight. To remove the influenee of bending, the strains in each member were averaged among the strains recorded by each of the devices on the member. For example, four strain readings at the corners of U1-L1 were averaged to obtain the axial force in that member. This was repeated for positions of the truck at intervals along the span. The resulting experimental influence lines were then plotted with those determined from structural analysis.

4.3. Findings

This section explains how both structural analysis and experimental load testing results demonstrate that the Lower Bridge at English Center carries coneentrated loads through truss action. The simple two-dimensional analytical model was successful in capturing the general behavior of, and in most cases approximating the axial forces in, a real three-dimensional system.[88] However, some aspeets of the real structure, such as sources of non-linear behavior, were not accounted for in the model.

[88] The analytical models consider only axial forces resulting from concentrated loads applied to panel points. In actuality, loads are applied to the deck and transferred through the floor system of stringers, beams, and girders. A load applied between panel points causes bending in the floor system, but can be resolved into a combination of panel-point loads.

4.3.1. Axial Force Results

Figs. 10a and 10b show axial forces occurring in models A and B when subjected to their corresponding dead loads. The dead loads, as shown in Table B-3, are essentially uniform loads and approximately equal for the two models. In the fully trussed Model A, the girder has significant axial forces that increase from the ends toward mid-span. Forces in the eye-bar chain increase dramatically from mid-span toward the towers. It may be said that such behavior is that of an inverted two-hinged trussed arch. This would be the behavior of the Lower Bridge at English Center for a uniform live load. The bridge would carry its dead load in this way only if the diagonals were tightened prior to removal of the falsework. Fig. 10b shows the forces in Model B for essentially the same uniform dead loads. These forces would exist in the English Center bridge under its own dead load if the falsework was removed prior to tightening the diagonals. The girder has negligible axial force and the forces in the eye-bar chain links are much more uniform. In fact, the horizontal component of the forces in the eye-bar links is constant, and the increase is due to the increasing slope of the links toward the towers. This behavior is that of a classical deck-stiffened suspension bridge.

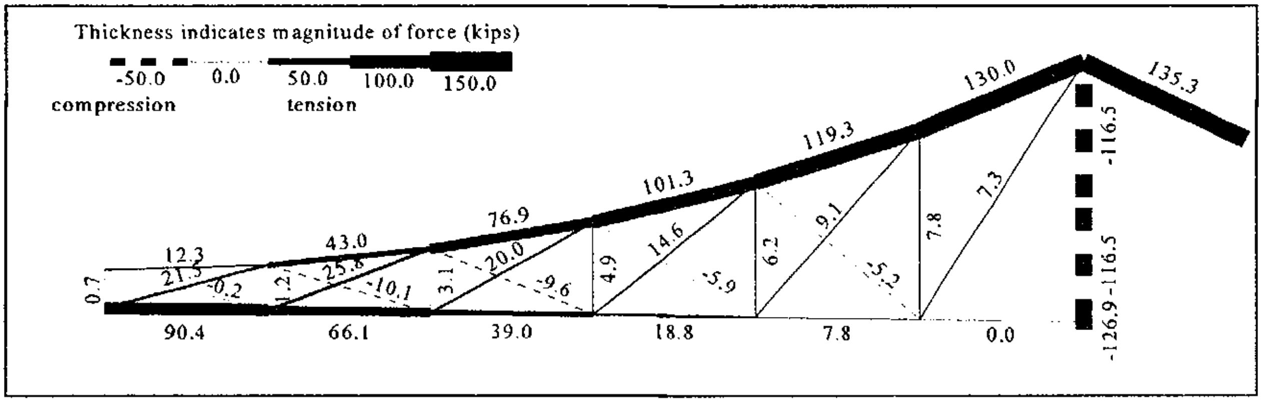

Figure 10a Axial forces in Model A under dead load.

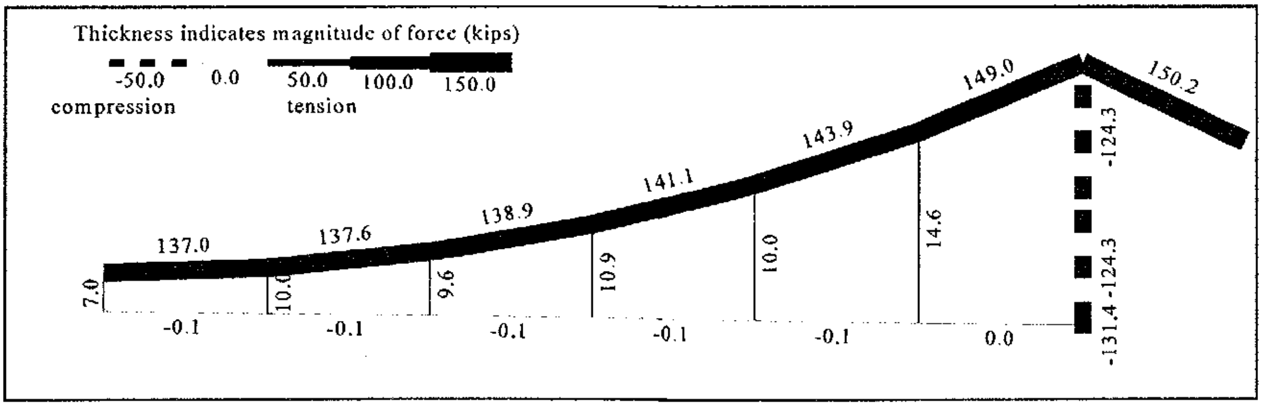

Figure 10b Axial forces in Model B under dead load.

Figs. 11a and 11b show member axial forces in the two models due to a 1-kip live load acting downward at mid-span. In Model B, the concentrated load produces relatively uniform tensile forces in all the vertical members (Fig. 11b). In fact, with equal panels, were the eye-bar chain exactly a parabola, force equilibrium in the original (undeformed) geometry would require that all of the vertical members have exactly the same force. This is true for any non-uniform live load. Fig. 11a shows that Model A carries the 1-kip live load very differently. The girder is again engaged axially and the eye-bar chain members on each side of the load have compressive forces. It can be said that the load is carried locally by truss action with the lower chord (girder) in tension and the upper chord (eye-bar chain) in compression. This observation is valid for all positions of the live load.

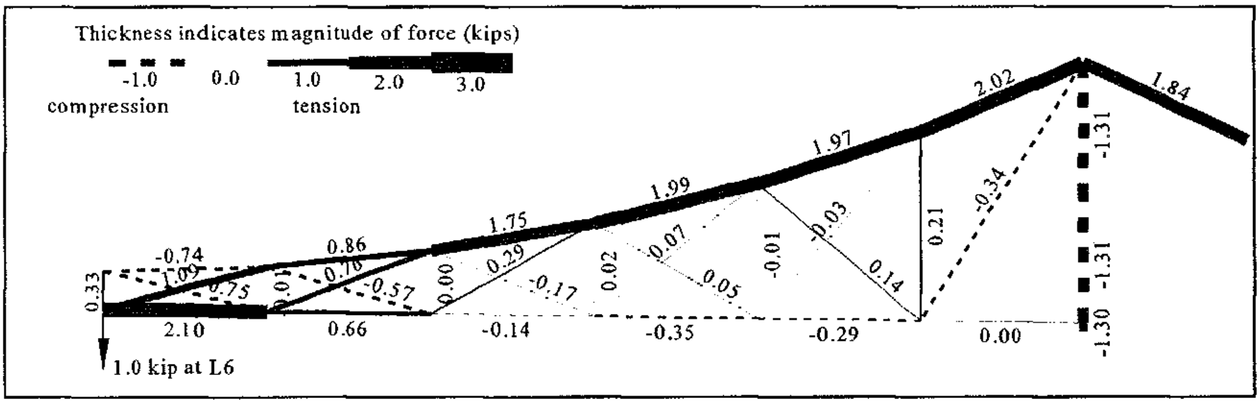

Figure 11a Axial forces in Model A with 1-kip live load at mid-span.

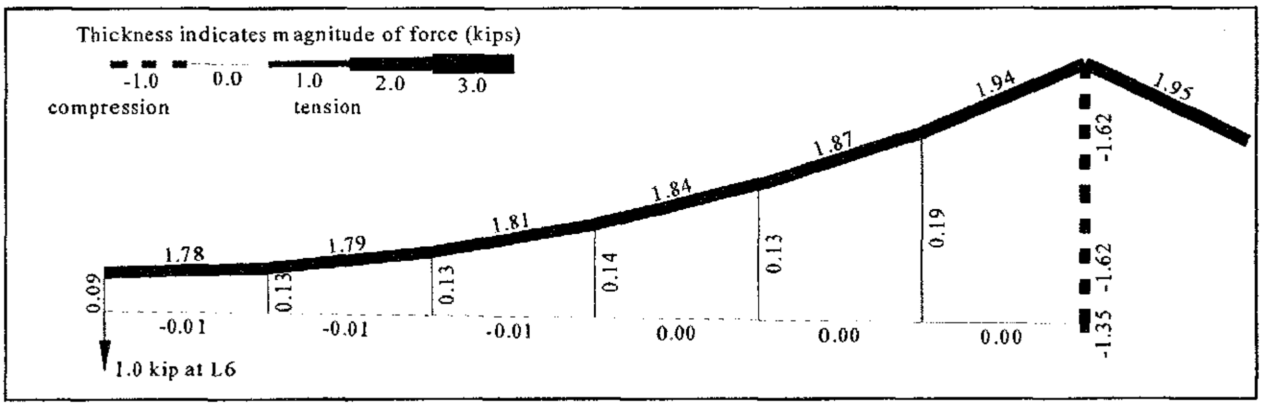

Figure 11b Axial forces in Model B with 1-kip live load at mid-span.

4.3.2. Deflection Results

During load testing of the bridge, the 18.6-kip truck load at mid-span (panel point L6) caused a vertical deflection of ahout 0.06 feet or 0.72 inches. This measurement was made with surveying equipment from PennDOT Maintenance District 3-2, placed upstream from the bridge on the north bank of Little Pine Creek. Analytical results for Model A give a vertical deflection of 0.09 inches under a unit load at mid-span. When multiplied by 9.3 kips (the load per truss), the analytical displacement is 0.86 inches, indicating that Model A is not as stiff as the actual bridge. Results from additional analytical models (Table 1) show that factors such as bending resistance of the deck system, partial joint fixity, and prestressed diagonals each decrease the mid-span deflection. Although the actual contributions of these factors to the English Center bridge's vertical stiffness is not known, its behavior seems to depend more on prestressing than on the other two factors.

Table 1 Mid-span deflection results for Lower Bridge at English Center and analytical models.

	Mid-span deflection (inches)		Remarks
	1-kip load at mid-span	9.3-kip load at mid-span*	
Load test	0.077	0.72	Actual deflection under 18.6-kip truck load (9.3 kips per truss).
Model A	0.092	0.86	Model assumes prestressed diagonals and slotted girder ends.
Model A-D	0.071	0.66	Ten-fold increase in girder stiffness reduces deflection 23 percent.
Model A-P	0.087	0.81	Deflection decreases 5 percent when girder ends are pinned.
Model A-Z	0.155	**	Without prestress in diagonals, deflection increases 68 percent.
Model B	1.070	9.95	Large deflection without diagonals indicates need for stiffer girder.

Notes:

* Except for the load test, 9.3-kip deflections were obtained by scaling up results from unit load analyses.

** Results from Model A-Z cannot be scaled up; see Section 4.3.3.

4.3.3. Influenee Lines

Influence lines (Figs. 12 through 18) show the variation of member force with load position for several members of interest. Somc differences between the experimental and analytical results are observable, revealing the limitations of analytical models. Model A does not consider the effects of non-linear behavior of connections (especially between the girdcr and the tower), diagonals becoming slack, or the floor system's contribution to effective girder stiffness. Models A-P and A-Z explored the effects of the first two factors separately. The influence lines from Model A-Z *cannot* be scaled or superimposed, however, because the buckling of any diagonal depends on the location or magnitude of the applied load. Results from

Model A-Z are included to show how Model A's behavior *under unit loading* would differ if all of its diagonals carried zero initial tension.

The influence lines do not show results from Model A-D, which explored the bridge's sensitivity to lower chord stiffness. The bridge carries concentrated live loads by truss action, which relies on axial, not bending, stiffness. As a result, the bridge's behavior is relatively insensitive to even a ten-fold increase in the lower chord's bending resistance. Results for Model A-D are therefore similar to those from Model A, and have been omitted for clarity's sake.

In most cases, the analytical results follow the same general trends as the experimentally obtained influence lines. Both sets of results confirm that the Lower Bridge at English Center acts as a truss for non-uniform loads. Furthermore, analytical and experimental results agree on points that are subtle and possibly counterintuitive. The truss-like behavior, sometimes contrary to that expected from a suspension bridge, includes force variation among the vertical members and occasional compression in these "hangers." One might be surprised to find that, upon applying a downward load to panel point L1 or L11, the vertical above it carries a compressive force. Analytical results demonstrate that the diagonals ascending from these panel points, not the vertical "hanger," carry the load to the eye-bar chain. Full trussing between deck and cable short-circuits the chain's ability to distribute loads between hangers as would the parabolic cable in a conventional deck-stiffened suspension bridge. That the hangers are built-up members rather than slender rods indicates that the bridge's designers may have anticipated this behavior.

The analytical models predict compression in the box-section vertical U1-L1 for a concentrated load at the first three panel points (see Fig. 12). The experimental results confirm this prediction, showing compression in vertical U1-L1 until the truck passes panel point L4, and justify its design as a compression member. Although models A and A-P underestimate the actual compressive forces, the shape of the analytical influence lines strongly resemble that of the experimental line. Results from Model A-Z, which more closely match those from load testing, indicate that prestressing of diagonals (or lack thereof) has a substantial effect on the forces carried by this vertical member.

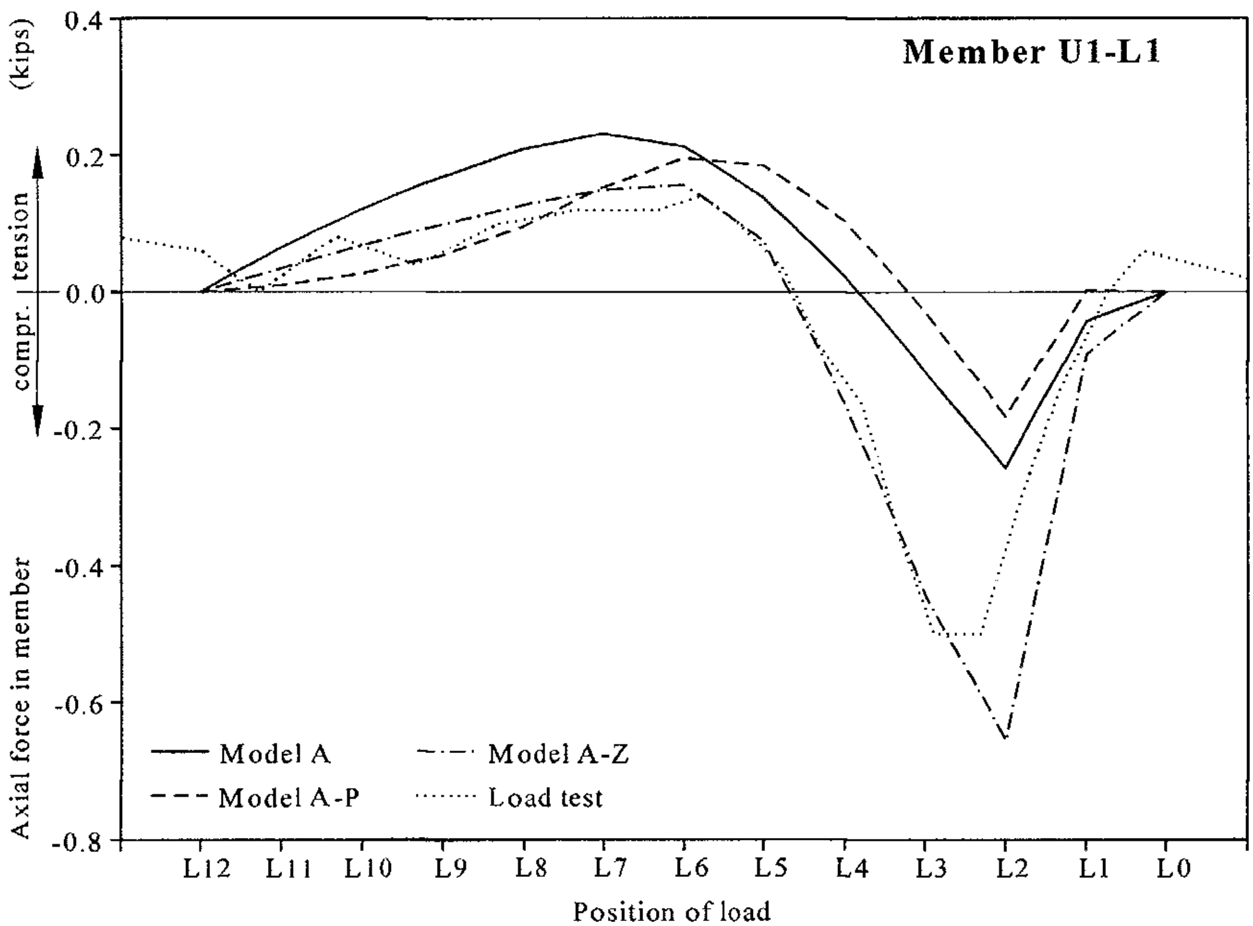

Figure 12 Influence lines for member U1-L1.

Analytical and experimental results for diagonal U0-L1 are better matched (see Fig. 13). This member is vital to the bridge's truss-like behavior, as is indicated by the relatively high forces observed in analysis and experiment. In Model A, for instance, diagonal U0-L1 carries a large portion of loads applied near the tower (a vertical component of 0.61 kips for a unit load at L1, 0.44 kips for a load at L2, etc.). As a load approaches the opposite end of the span, all results (except those for Model A-Z) show significant compressive forces in the member. This large range of cyclic stresses makes the member subject to fatigue failure.[89] In fact, the fractured ends of diagonals U0-L1 and U12-L11 show that these members failed at least once during the bridge's lifetime and that their size was increased from the original design. The compressive forces shown by analysis are certainly above this slender member's buckling limit (note that results from Model A-Z show zero force for unit loads applied past L5). Experiment confirms that diagonal U0-L1 does undergo compression, or more accurately, a reduction of the initial tension that must be present in that member.

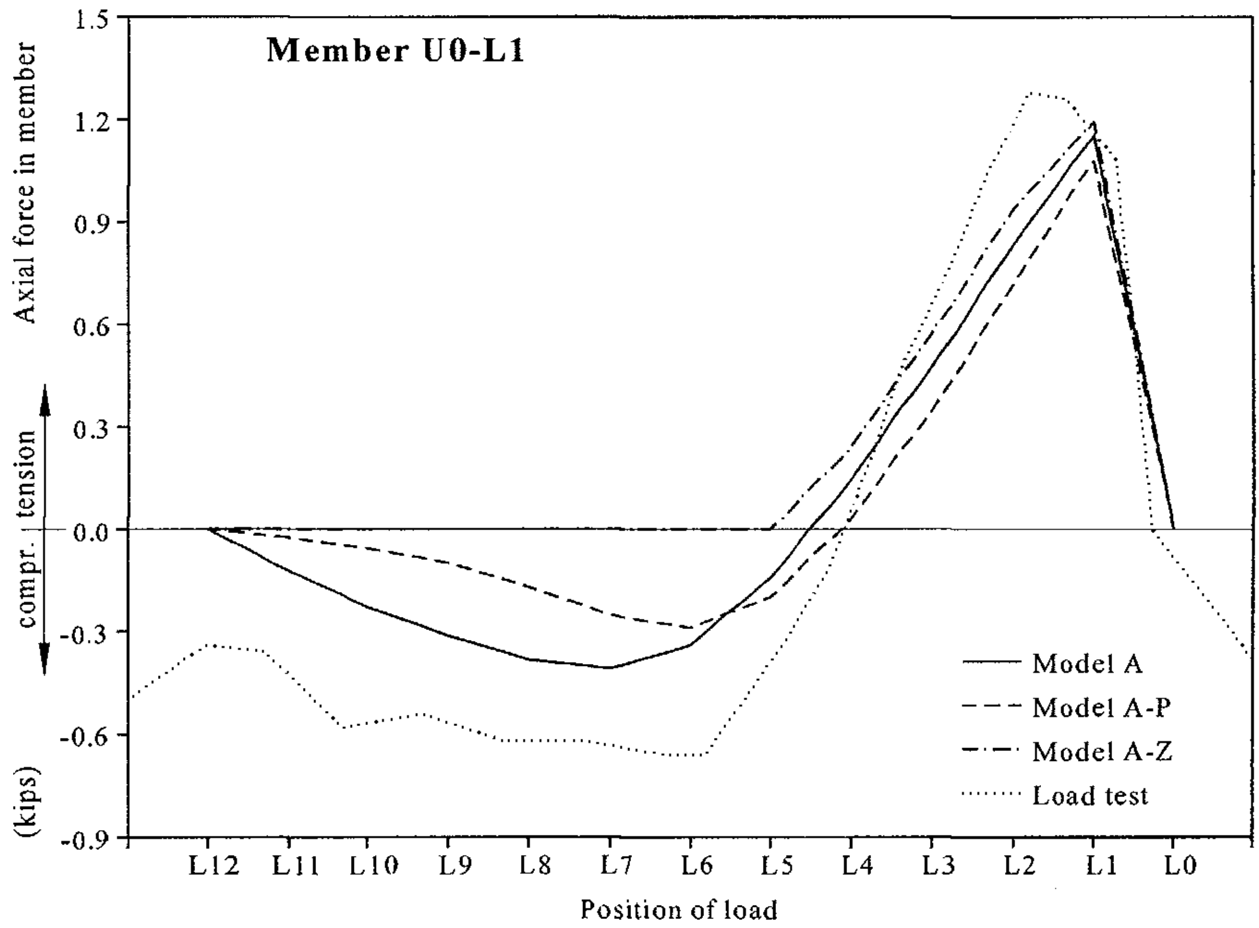

Figure 13 Influence lines for member U0-L1.

[89] See "Fatigue Life Estimate" appended to Lichtenstein, "BMS No. 41-4001-0270-0000."

Influence lines for member L0-L1, the longitudinal girder segment framing into the tower, illustrate the limitations of the analytical model (Fig. 14). Models A and A-Z contain an idealized (frictionless) slotted connection at joint L0, so member L0-L1 carries zero force for any position of the load. Because of the bridge's camber, the lower chord carries compressive force like a two-hinged arch when its ends are pinned in Model A-P. None of the models can capture the connection's real behavior, which is a dynamic, non-linear sequence of sliding and sticking. In the actual English Center bridge the longitudinal girder's slotted ends are neither frictionless nor totally pinned, so its behavior should fall somewhere between that of the two extremes. The experimental influence line shows that this is indeed the case.

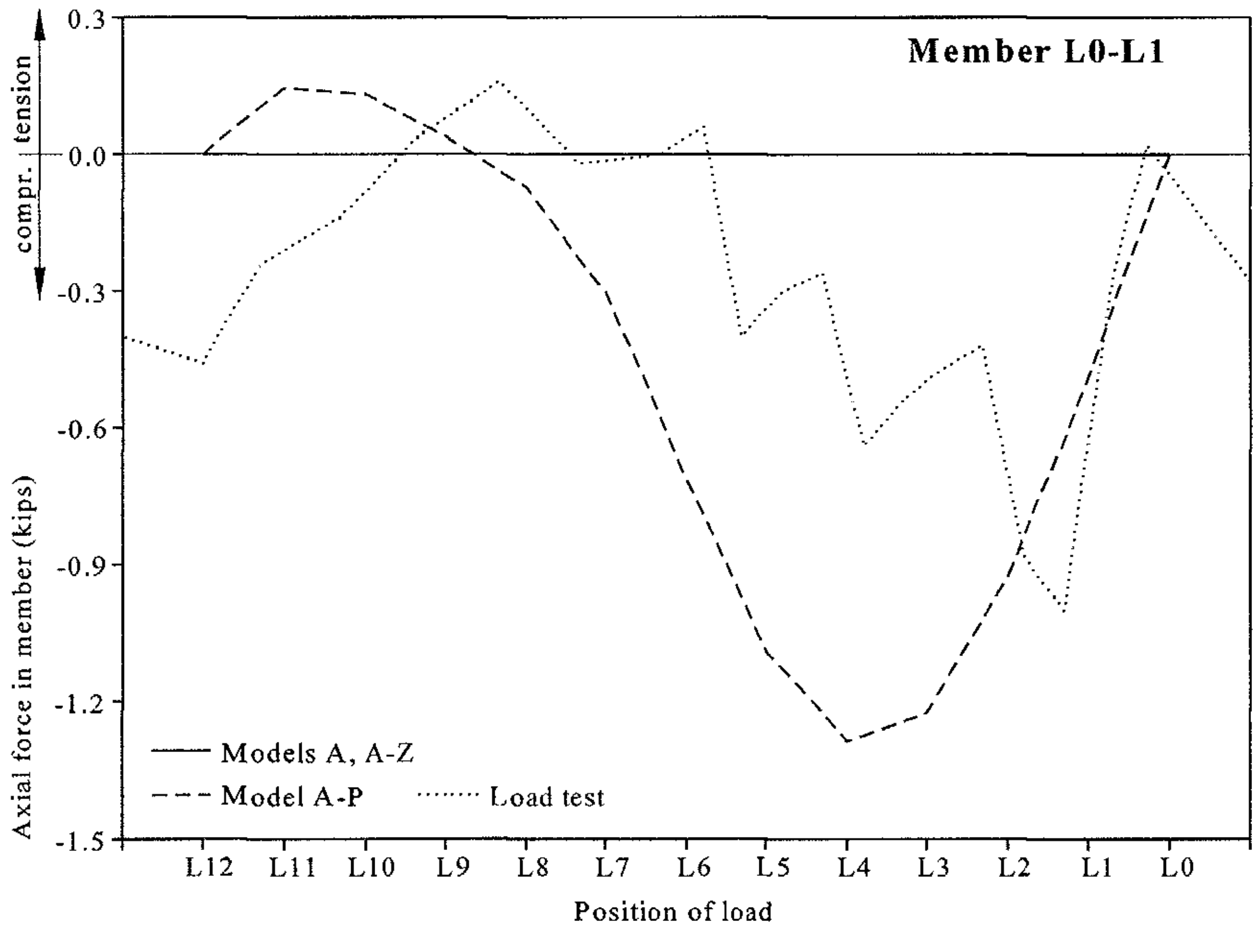

Figure 14 Influence lines for member L0-L1.

Structural analysis of several other members produced influence lines which closely match those from load testing (Figs. 15 through 17). Eye-bar link U1-U2 carries almost exclusively tensile forces, consistent with its role as the upper chord of an inverted trussed arch (see Fig. 15). Both analytical and experimental results, however, show compressive forces in link U5-U6 for concentrated loads applied near mid-span (Fig. 16). These compressive forces in link U5-U6, which also belongs to the inverted arch's upper chord, might seem counterintuitive. But especially where the truss is shallowest near mid-span, diagonals provide the load path of least resistance (greatest vertical component of structural stiffness) and therefore carry the most force from the loaded panel point.[90] When the diagonals are working in tension, the eye-bar links between them serve as a compressive strut. Comparing the influence lines for link U5-U6 to those for diagonal U5-L6 (Fig. 17), one can see that the eye-bar's peak compression occurs at approximately the same load position as the diagonal's peak tension. Results from Model A-Z for these two members differ substantially from those of the other analytical models and load testing, indicating that prestressing of diagonals has a significant effect on the forces carried by members near mid-span.

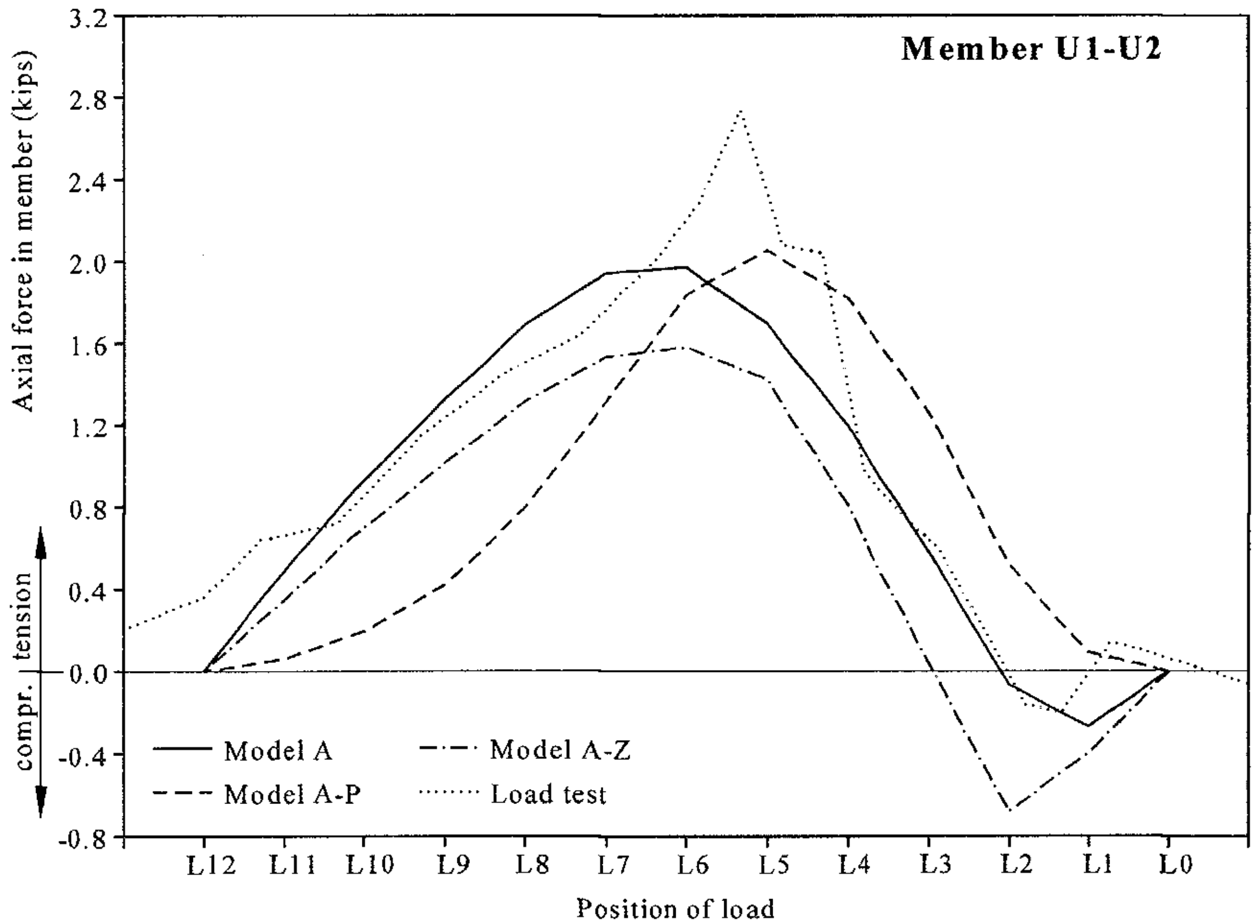

Figure 15 Influence lines for member U1-U2.

[90] As Fig. 11a shows, the vertical carries only 0.33 kips (33 percent) of the 1-kip load at mid-span, leaving the diagonals to carry the remaining 67 percent.

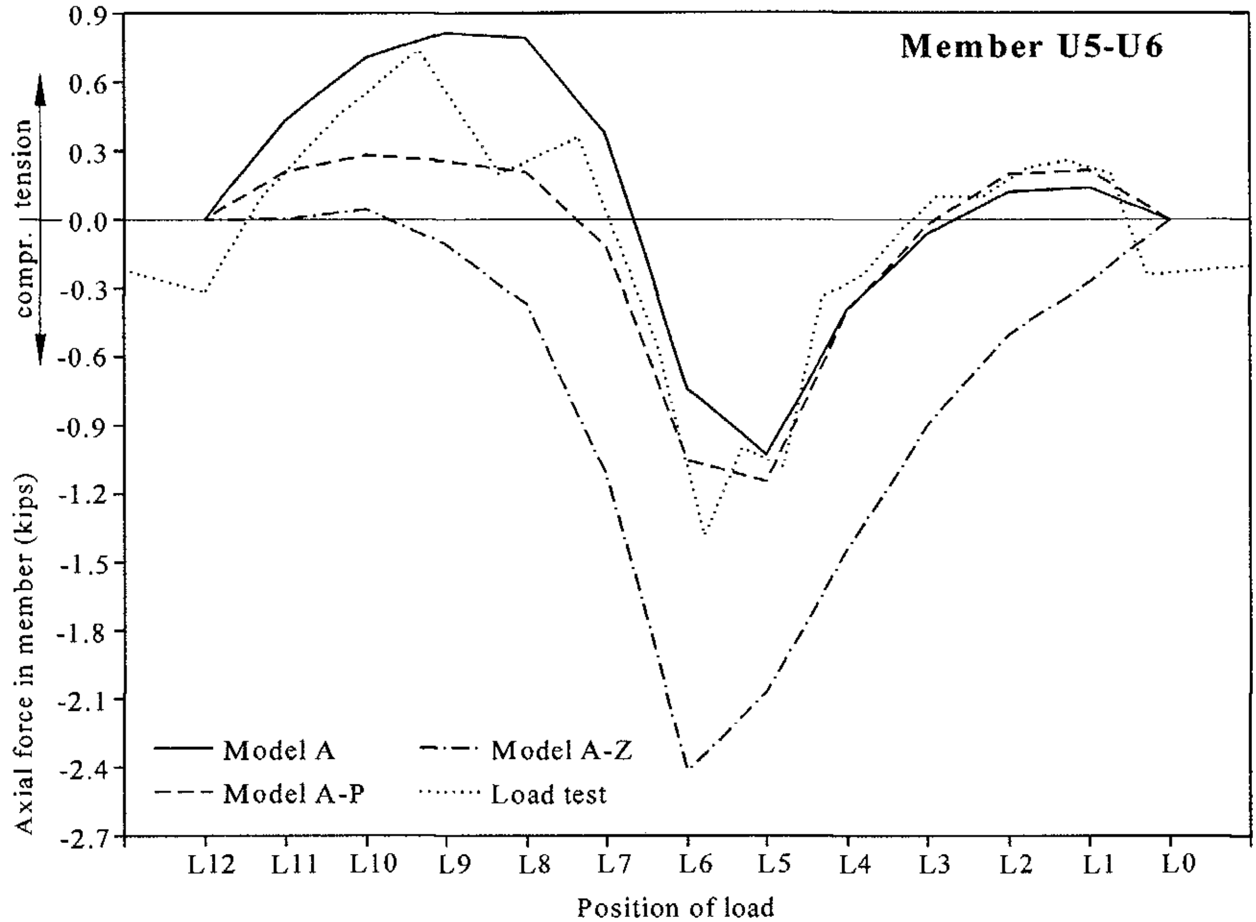

Figure 16 Influence lines for member U5-U6.

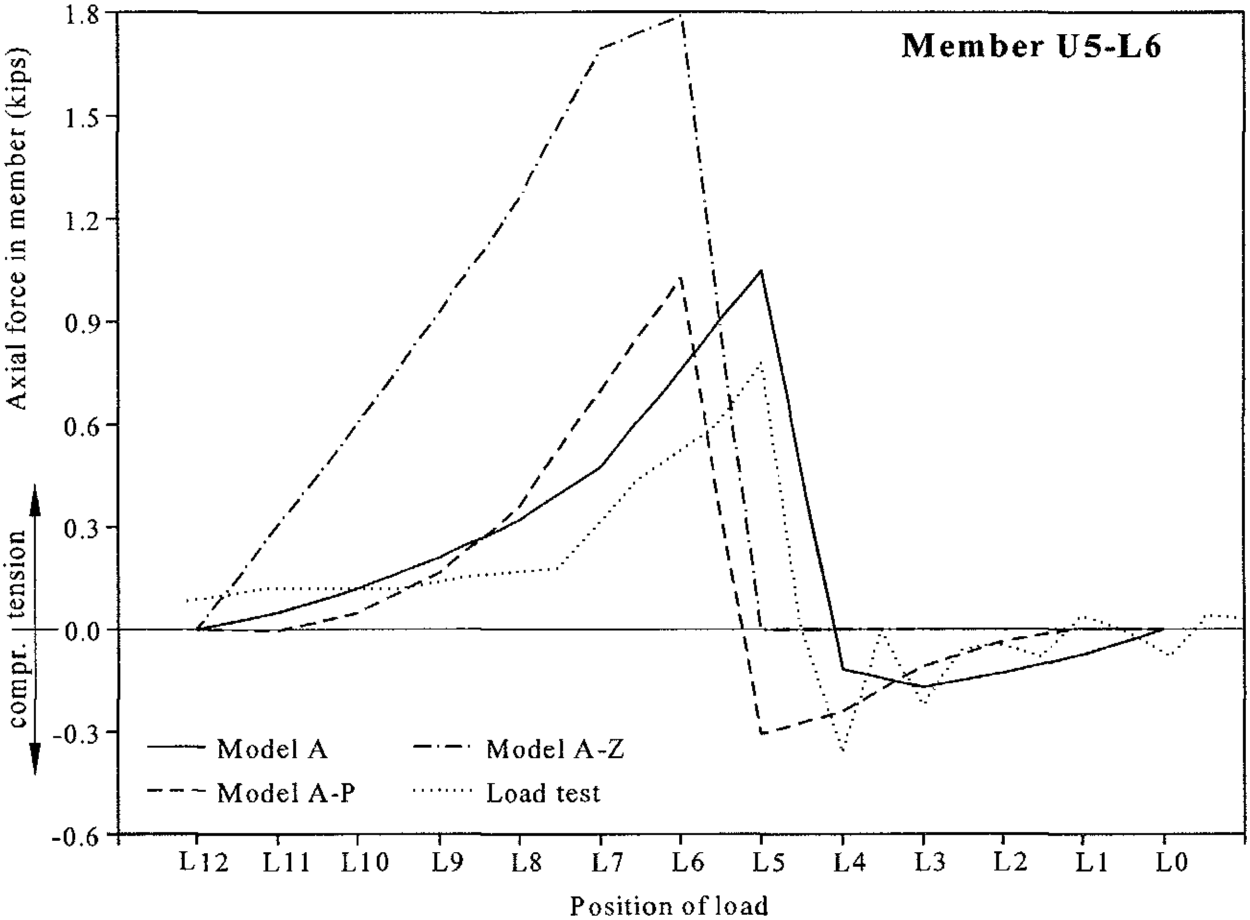

Figure 17 Influence lines for member U5-L6.

Finally, analytical and experimental results both show that the force in the backstay member, whose horizontal component resembles the horizontal thrust of an inverted arch, reaehes a roughly symmetrical peak near mid-span (see Fig. 18). Results from Model A-Z show that the force carried by the backstay varies little with the buckling of diagonals. Were the inverted arch truly two-hinged, i.e., horizontally fixed at the ends of its upper chord only, the influence line would be exactly symmetrieal as shown in the results for models A and A-Z. The peak shifts toward point L0 in Model A-P, and to a lesser extent in the load test, beeause of horizontal fixity in the girder-to-tower connection. Again, the idealized slot in models A and A-Z, and the pinned connection in Model A-P, bracket the actual English Center bridge's behavior.

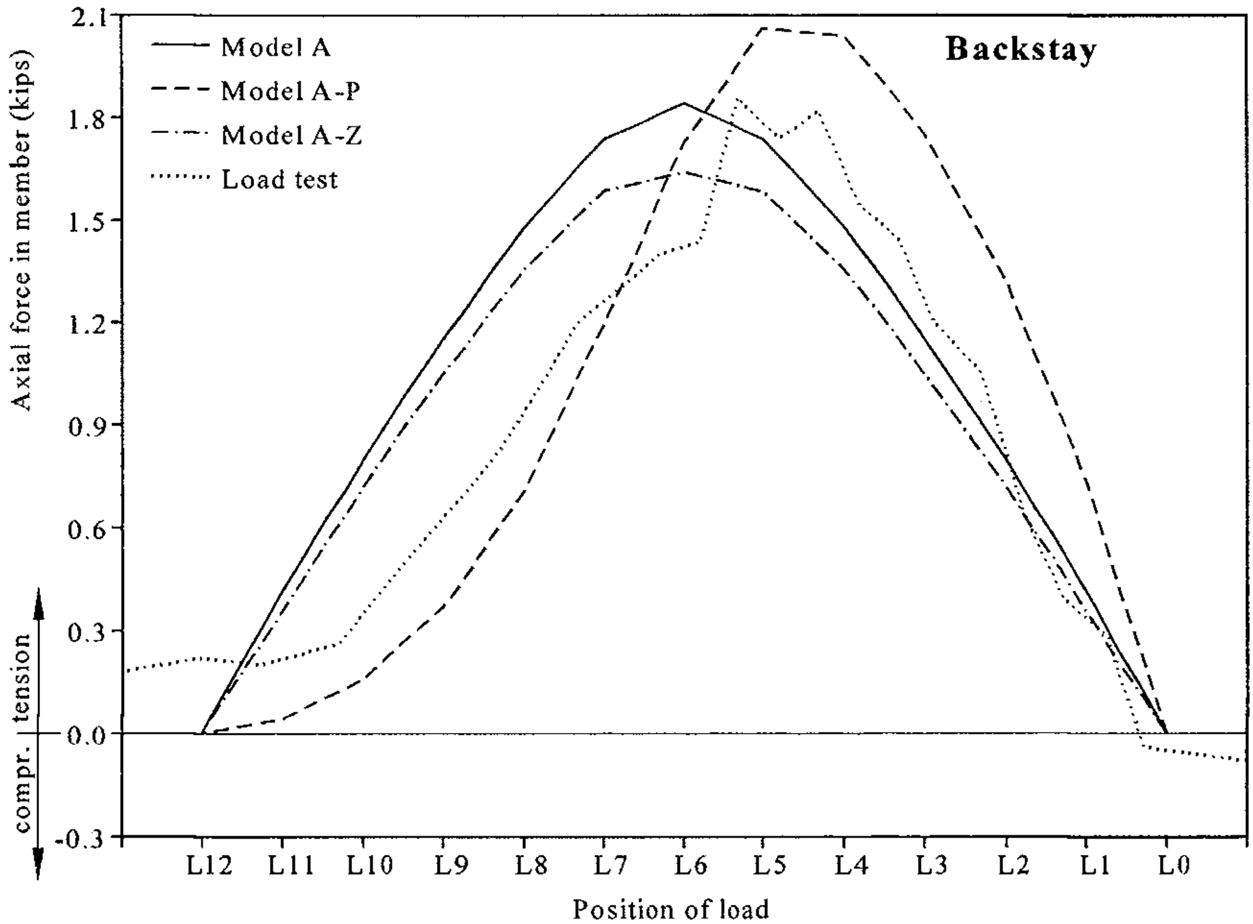

Figure 18 Influence lines for backstay member.

5. STATIC BEHAVIOR OF RELATED BRIDGE FORMS

5.1. The "Kit of Parts"

To gain further understanding of the Lower Bridge at English Center and other suspension bridge forms, six additional models were developed. These models, designated C through H and shown schematically in Fig. 19, share the same span length, support conditions, and member properties as the English Center bridge. One might visualize the modeling process by imagining a "kit of parts" (eye-bars, diagonals, and verticals) that can be assembled between the towers of the actual bridge into any number of suspension forms supporting the deck. Each of the models was subjected to its own dead load (listed in Table B-3), and also unit live loads at each panel point.[91] The results are discussed briefly following each model's description.

Models C through H were intended for comparing the behavior of stiffening systems that may be made to look the same — and therefore may be confused for one another. The six models were compared by rearranging the truss members into various forms and noting general patterns of axial forces resulting from common loads applied to each (Figs. 20 though 26). The results provide insight into qualitative differences in behavior, although, for the statically indeterminate models, specific member forces will be affected by the distribution of relative member stiffnesses. Design iterations to meet strength and serviceability criteria for each form were beyond the scope of this study of the Lower Bridge at English Center.

[91] Dead loads vary slightly among the models because not all of the parts were used in each.

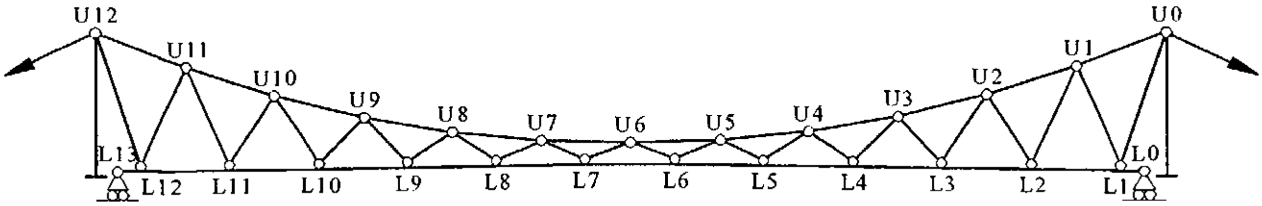

Figure 19a Model C — fully trussed two-hinged inverted arch with Warren triangulation.

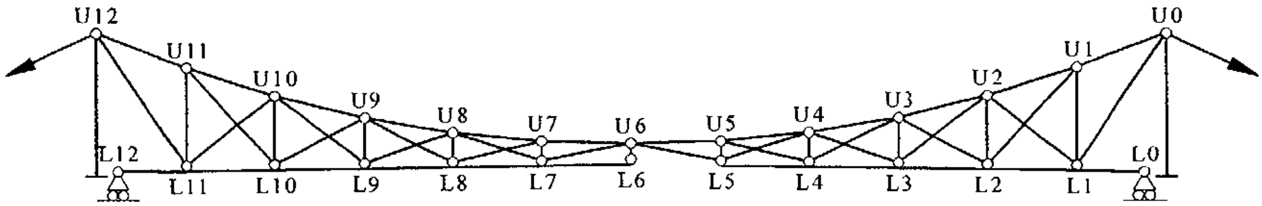

Figure 19b Model D — fully trussed three-hinged inverted arch.

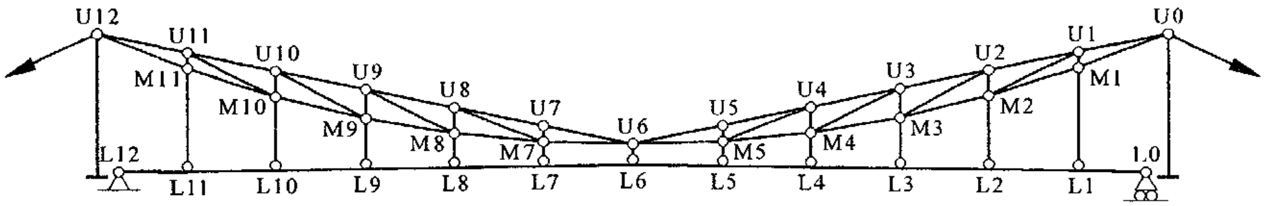

Figure 19c Model E — three-hinged inverted arch (braced chain) with hangers.

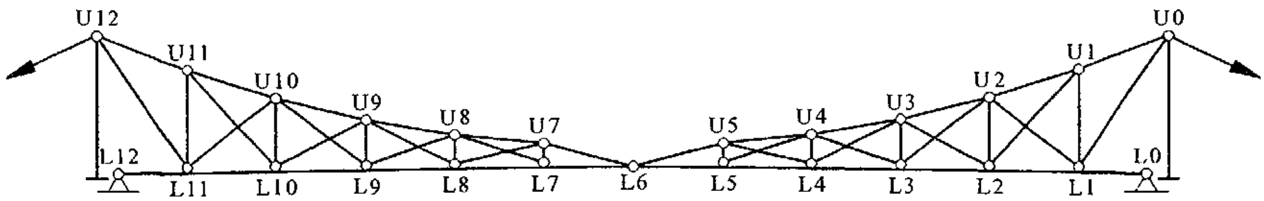

Figure 19d Model F — cantilever form with no suspended span.

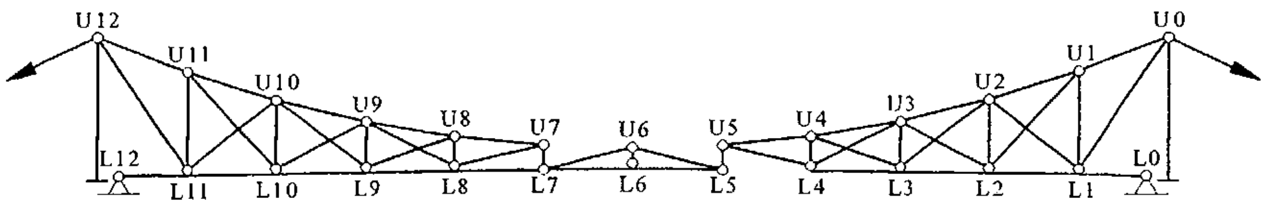

Figure 19e Model G — cantilever form with central supported span.

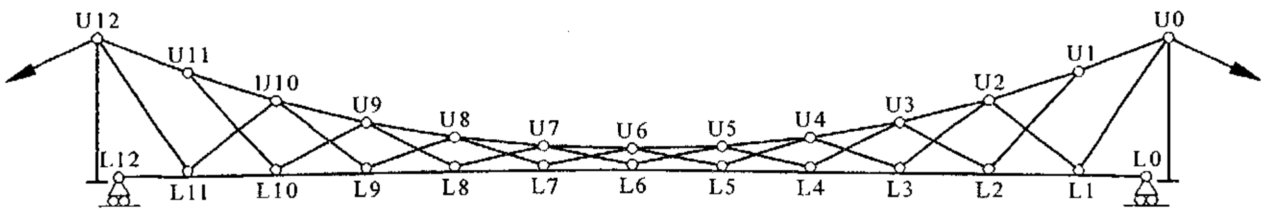

Figure 19f Model H — fully trussed two-hinged inverted arch without vertical members..

Model C is fully trussed between deck and eye-bar chain like the English Center bridge, but its trussing pattern is of the Warren type.[92] All web members were modeled with the looped eye-bar diagonal members, with their diameter increasing toward mid-span. Member forces from an analysis of this model under dead load are shown in Fig. 20a. The results show similar behavior to the Lower Bridge at English Center, with chain tension increasing toward the towers, tension in the bottom chord increasing toward mid-span, outward-leaning diagonals in tension, and inward-leaning counters in compression. (As with Model A, pretension is assumed to permit the slender diagonals to carry compressive forces.) For a load near mid-span, Fig. 20b shows that Model C's behavior is likewise similar to that of Model A, with the upper chord in compression directly above the load. Such similar behavior suggests that full trussing between cable and deck carries loads in this way, regardless of the trussing pattern.

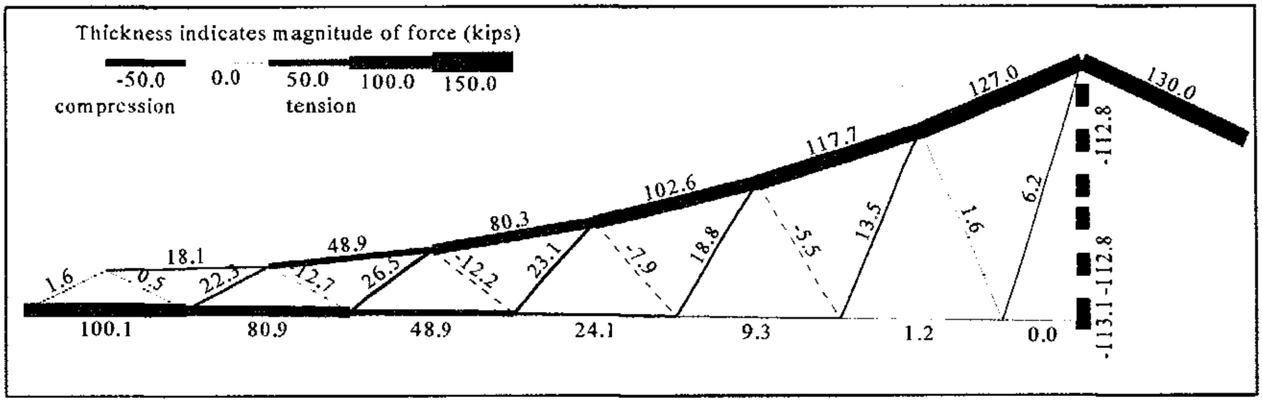

Figure 20a Axial forces in Model C under dead load.

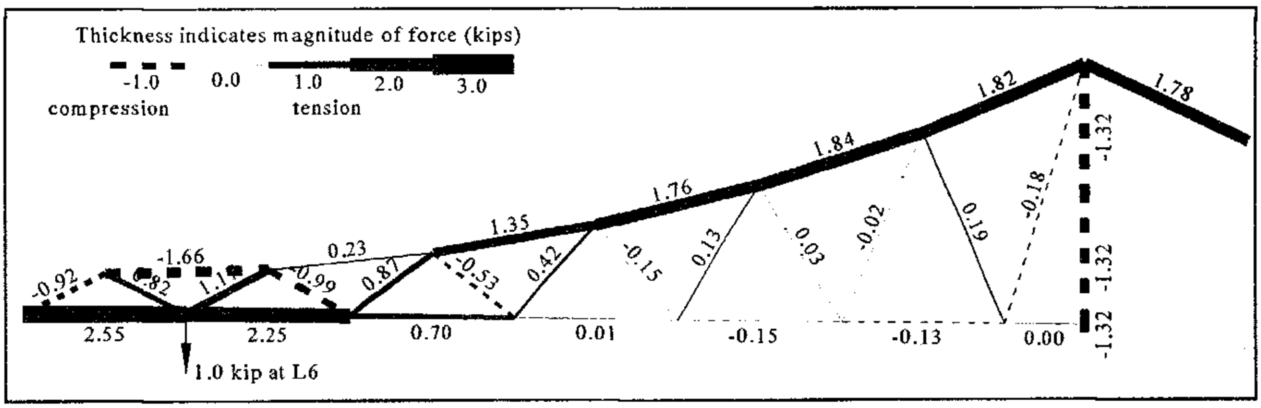

Figure 20b Axial forces in Model C with 1-kip live load near mid-span.

[92] The Warren-type trussing visually resembles the inclined stays of Freeman, Fox, and Partners' Severn and Humber bridges.

The Lower Bridge at English Center behaves as an inverted two-hinged arch, but a three-hinged form may also be defined. Model D has a third hinge, created by breaking the longitudinal girder at mid-span. Again, the box-shaped verticals were pinned at L1 and L11. Under dead load, the chain carries a nearly constant tensile force (Fig. 21a). Between the hinges, two half-span segments of structure behave as trusses, with their top chords in tension and bottom chords in compression for downward loads. Under a concentrated load at mid-span, the chord forces decrease with increasing truss depth toward the supports (Fig. 21b).

One might notice that for the right half of Model D shown in Figs. 21a and 21b, the truss' top chord consists entirely of eye-bar links while the effective bottom chord includes not only the longitudinal girder from L1 to L5, but also "kit of parts" diagonals U0-L1 and U6-L5. The cross-sectional area of the diagonals is much smaller than that of the longitudinal girder segments, which, combined with their greater length, results in a lower axial stiffness and thus greater contraction under compressive forces. This is particularly significant at mid-span, where they cause a large local deflection, making Model D appear significantly less stiff than Model A. Further design iterations of Model D could increase its overall vertical stiffness by replacing the slender diagonals U0-L1 and U6-L5 with members of greater cross-sectional area.

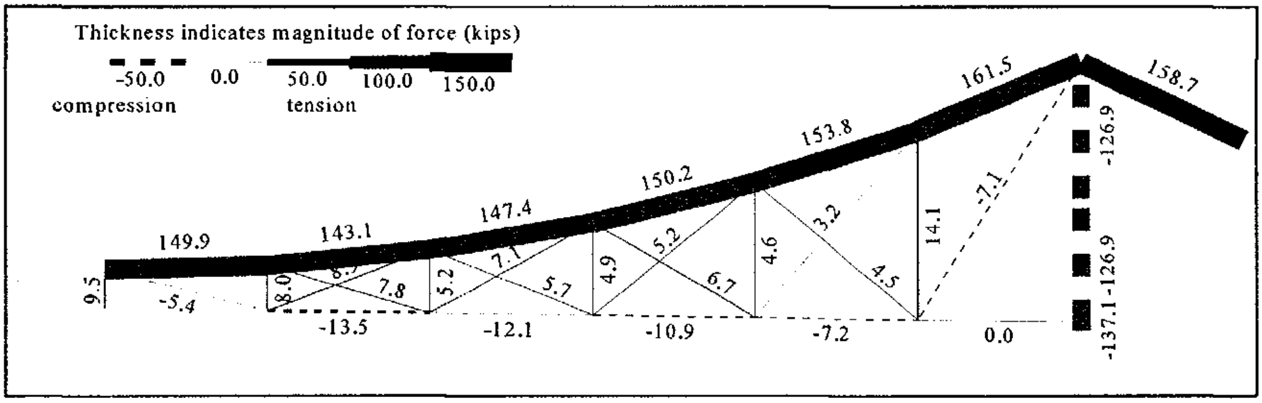

Figure 21a Axial forces in Model D under dead load.

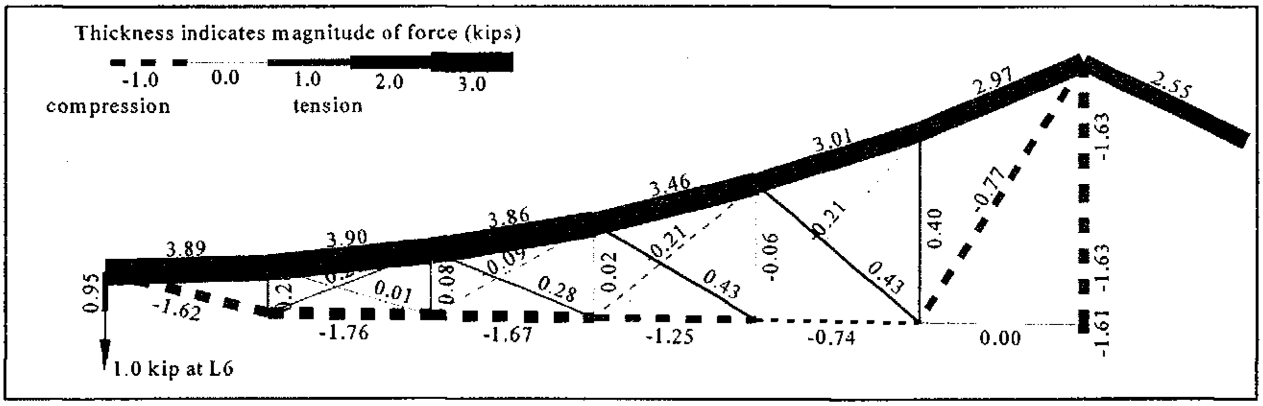

Figure 21b Axial forces in Model D with 1-kip live load at mid-span.

Model E resembles thc three-hinged braced-chain form used by Hemberle for the Point Bridge in 1876. In this model, the braced chain's lower (suspension) chord follows the curve of the English Center bridge's top chord; the braced chain's upper (stiffening) chord follows a straight line from the top of each tower to mid-span. Again, all verticals were pinned to the longitudinal girder. In this re-arrangement of thc "kit of parts," top chord cye-bars were divided equally between the suspension and stiffening chords. (Real braced-chain bridges, however, typically have a larger suspension chord and a smaller stiffening chord. Because the verticals act as hangers below the chain, one end of the girder was pinned to restrain the deck longitudinally. Member forces in Fig. 22a show that Model E's dead-load behavior is somewhat similar to a conventional suspension bridge, having a nearly constant tensile force along the suspension chord and low axial forces in the deck. The stiffened chain in Model E distributes loads among the suspenders, but not equally, as would a (geometrically linear) parabolic chain. Mid-span hanger U6-L6, for instance, carries an unusually high axial force — almost twice the dead load applied to panel point L6. This is beeausc the stiffening chords lie along straight lines from U0 to U6 and from U6 and U12, which is the funicular polygon for a concentrated load at mid-span. Largc shears and bending moments in the lower chord at panel point L6 (3.64 kips and 69.6 kip-ft for the dead load casc) confirm that the stiffening chords create a relatively stiff zone, or effective vertical "support," at mid-span. This effect is evcn more prominent for a concentrated load at mid-span, in which case thc stiffening chords carry most of the load (Fig. 22b).

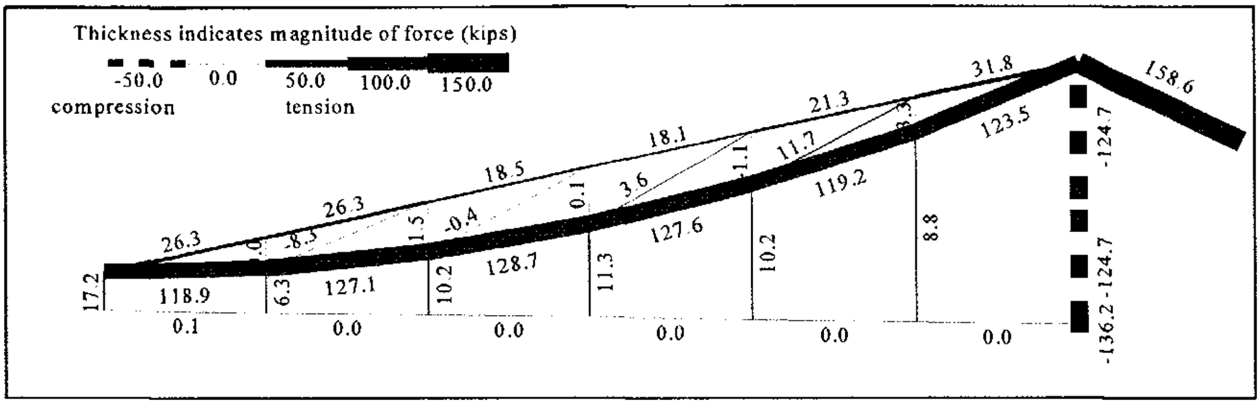

Figure 22a Axial forces in Model E under dead load.

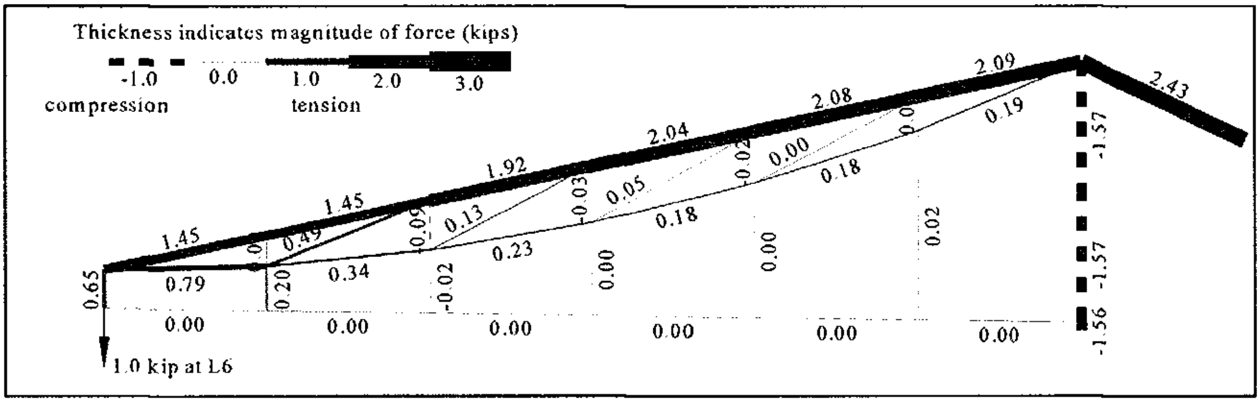

Figure 22b Axial forces in Model E with 1-kip live load at mid-span.

Other load cases show that Model E's behavior is that of an inverted three-hinged trussed arch. Under dead load, the stiffening chords carry smaller forces than the suspension chords, the fraction increasing from roughly one-sixth at panel points U3 and U9 to about one-quarter at the hinges. Diagonals attached to panel points U1, U2, U10, and U11 are in tension; and those attached to U3, U4, U8, and U9 are in compression; as is expected for web members in a three-hinged trussed arch loaded uniformly. Under moving loads, the trussed arch's chord members experience stress reversals. For a concentrated load at mid-span, both stiffening and suspension chords are in tension, with the stiffening chord carrying most of the load. When the concentrated load is applied asymmetrically, the loaded side's suspension chord is in compression and its stiffening chord in tension (Fig. 23a); the opposite applies to the unloaded side (Fig. 23b). Member forces on the loaded side are noticeably higher as a result of the effective "support" at mid-span. As with previous models, the addition of trussing causes variation of forces in the chain.

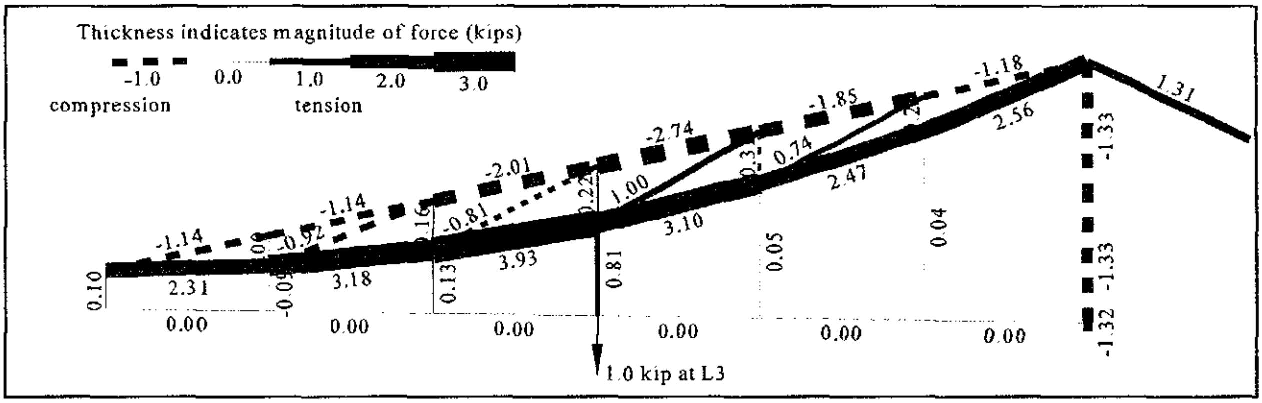

Figure 23a Axial forces in Model E with 1-kip live load at quarter point L3.

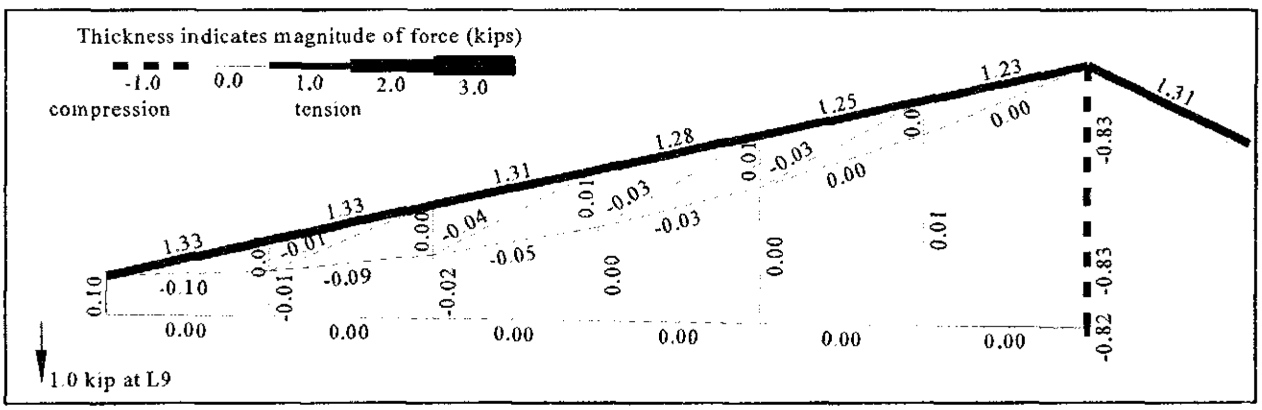

Figure 23b Axial forces in Model E with 1-kip live load at quarter point L9.

To address Victor Darnell's comment that the English Center bridge "might be a double cantilever," Model F was created by deleting the top chord at mid-span to form a cantilever bridge with no supported span.[93] Since the resulting model is a cantilever, both ends of the longitudinal girder must have pinned connections. Again, all verticals were pinned to the longitudinal girder. Fig. 24a shows that under dead load, the top chord's tensile force increases toward the towers. Diagonals are in tension, and counters in compression, as is expected in a cantilever arm. The bottom chord is in tension at mid-span, but then has increasing compressive force toward the tower. Obviously, the slotted girder-to-tower connection in the actual Lower Bridge will not resist compressive forces until the bolts bear on the slot's far end. Though this did occur in the bridge during load testing, it is not the structure's intended means of carrying load. The Lower Bridge's top chord is continuous at mid-span so it can help carry loads — it is not there simply to disguise a cantilever hinge. With a concentrated load at mid-span, Model F behaves similarly to Model A: some compression in the bottom chord, stress reversals in the diagonals and counters, and a large portion of the load carried by the eye-bar chain (Fig. 24b).

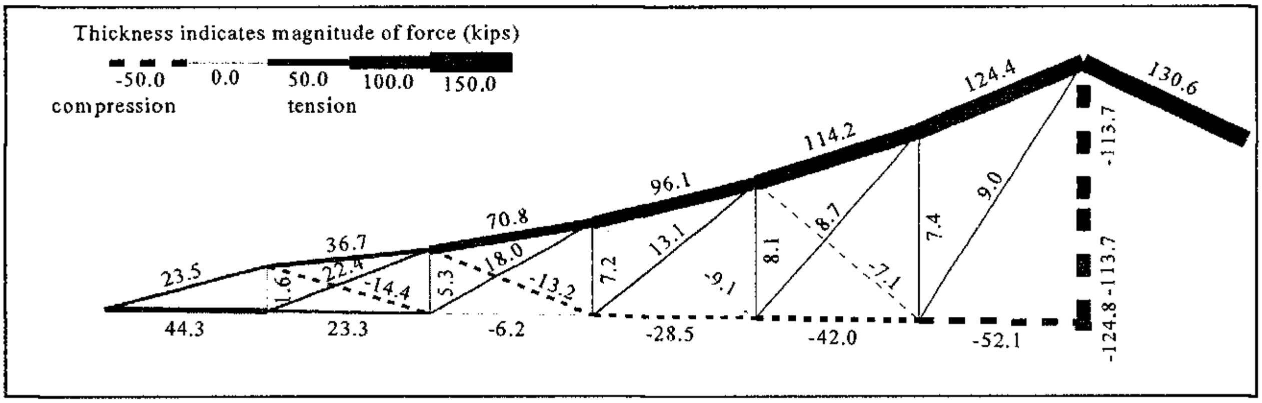

Figure 24a Axial forces in Model F under dead load.

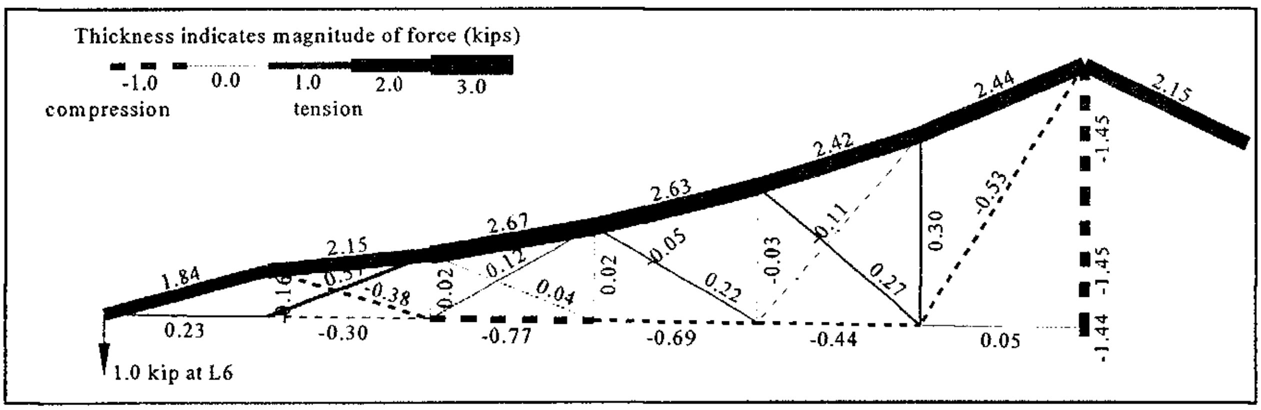

Figure 24b Axial forces in Model F with 1-kip live load at mid-span.

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[93] HAER No. PA-461, "Lower Bridge at English Center," 7.

Darnell also compared the English Center bridge to the Northampton Street Bridge in Easton, a cantilever bridge with a short supported span made to resemble a suspension bridge by inserting non-structural eye-bars.[94] Model G was created from the "kit of parts" by deleting the top chord at mid-span, deleting diagonals, and breaking the longitudinal girder to form a simply supported king-post truss at mid-span. All verticals were pinned to the longitudinal girder, and both girder ends were pinned. Because this model is essentially Model F with shorter cantilever arms and a supported span its behavior is very similar, under both uniform and concentrated loads (Figs. 25a and 25b).

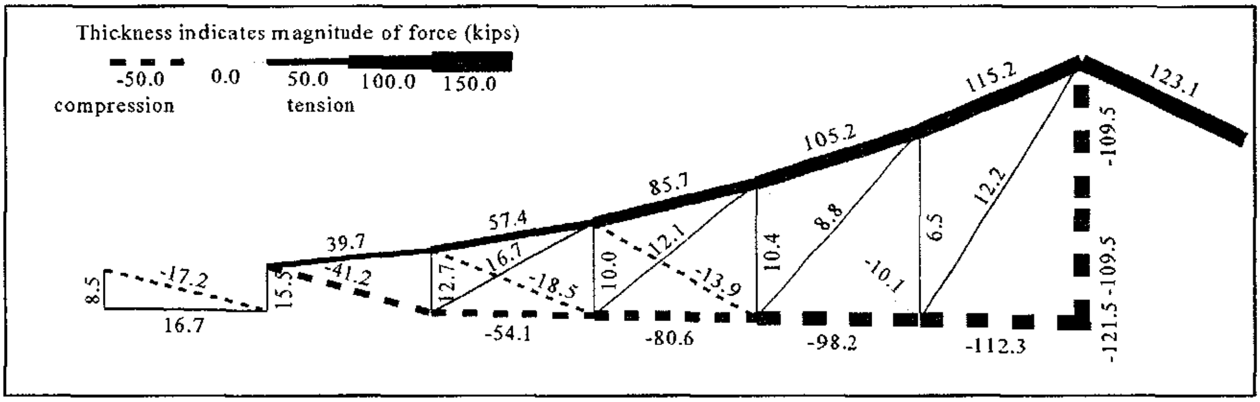

Figure 25a Axial forces in Model G under dead load.

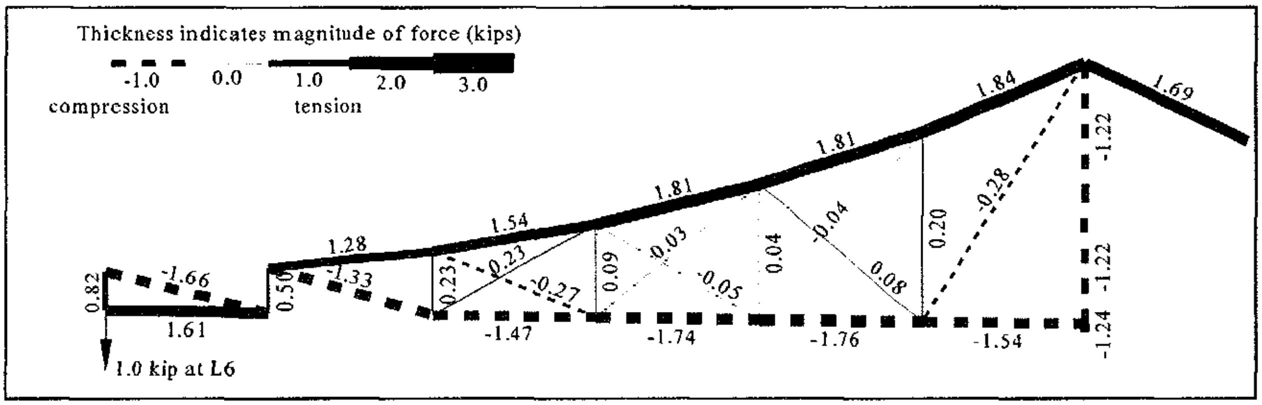

Figure 25b Axial forces in Model G with 1-kip live load at mid-span.

[94] HAER No. PA-461, "Lower Bridge at English Center," 7.

Finally, Model H was created by deleting the verticals from the Lower Bridge at English Center. Creation of this model was motivated by the remarks of engineer Harry J. Engel, written on a bridge inspection drawing,

> Since the actual verticals are not suspenders, having at their bases only four 5/8" [diameter] rivets each in tension to attach them to the top Ls of the longitudinal girder, the diagonal rods into the panel points carry these [floor] loads.[95]

Both longitudinal girder ends were modeled as rollers. Comparing Figs. 10a and 26a, the forces in Model H under dead load are similar to those in Model A, especially in the chords. Similar forces exist in models A and H when a concentrated live load is applied at mid-span (Figs. 11a and 26b). Because the two models are so similar, with no general trend as to which produces higher forces, it seems that Engel's simplification produces a reasonable statically determinate model for determining chord forces. However, this does not account for the bridge's design, in which the verticals were obviously proportioned (differently) to carry compressive loads.

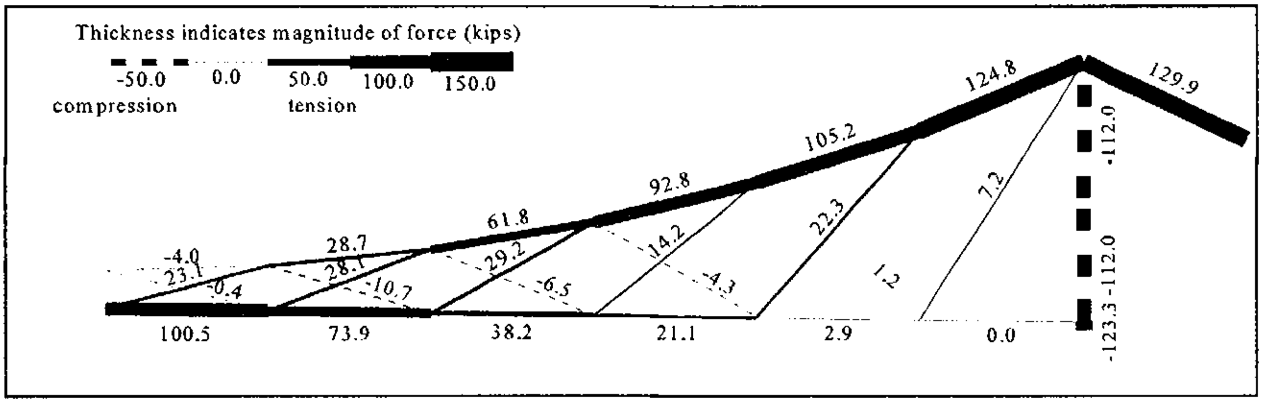

Figure 26a Axial forces in Model H under dead load.

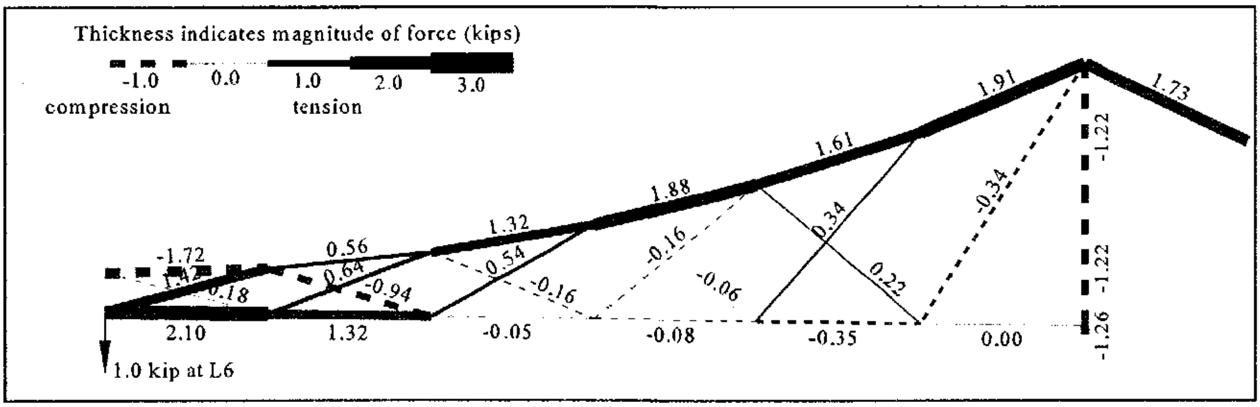

Figure 26b Axial forces in Model H with 1-kip live load at mid-span.

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[95] Engel, "Inspection Drawing."

5.2. Static Non-Linear Behavior of Deck-Stiffened Suspension Form

In models A, C, D, F, G, and H, the various arrangements of diagonal bracing allow for truss action, with the suspension chain acting as an upper chord and the longitudinal floor girder as a lower chord. This effective truss provides substantial bending stiffness to the bridge, thereby limiting displacements due to live loads. In Model B, however, the longitudinal floor girders provide the only bending stiffness to the bridge. Live-load displacements may therefore be significantly larger for the same span and live loading than those which would occur in the fully trussed system of Model A (see Table 1). In order to properly account for the larger live-load displacements expected in Model B, a geometrically non-linear analysis is required. A geometrically non-linear analysis satisfies conditions of equilibrium and compatibility on the deformed shape of the bridge, whereas the linear analyses used elsewhere in this study assume the displacements to be negligible compared to the overall dimensions of the structure.

Unstiffened suspension bridges have long been recognized as susceptible to large displacements under certain loading conditions. In his *Rapport a Monsieur Becquey,* Navier developed geometrically non-linear relations to quantify the effects of live loads on "unstiffened" suspension bridges — bridges which have only a parabolic cable or chain and a road deck, but no stiffening girders or diagonal bracing systems.[96] By accounting for these non-linear effects, Navier's work introduced the concept of cable stiffness, whereby the tension in the parabolic cable caused by the bridge's dead load results in a vertical stiffness, or tendency to resist vertical displacements caused by live loads, as demonstrated qualitatively in Fig. 4. Major developments in the theoretical analysis of suspension bridges were later achieved by Josef Melan in 1888.[97] Melan published a rigorous analysis of suspension bridges stiffened with a longitudinal truss or girder, including the effects of interaction between the cable and stiffening girder. Melan's work included both linear and geometrically non-linear analyses. The linear theory has since become known as the Elastic Theory, and the geometrically non-linear analysis as the Deflection Theory.[98] The Elastic Theory is analogous to the linear stiffness method used by computer structural analysis software for the analyses presented in Sections 4.3 and 5.1.

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[96] Navier, *Rapport a Monsieur Becquey.*

[97] Josef Melan, "Theorie der eisernen Bogenbrücken und der Hängenbrucken," *Handbuch der Ingenieurwissenschaften* (Leipzig: Wilhelm Engelmann, 1888 and 1906). See also a translation by David B. Steinman, *Theory of Arches and Suspension Bridges* (Chicago: Myron C. Clark, 1913).

[98] For a more complete history of the development of suspension bridge theory, see Buonopane and Billington, "Theory and History."

Because a geometric non-linear analysis is more complex than a linear elastic analysis, a simplified version of Model B is used, shown in Fig. 27 and called Model B-NL hereafter. This simplified model introduces the following assumptions:

1. *Parabolic cable of 300'-0" span and 32'-4" sag* — The continuous curve of this parabola closely approximates the geometry of the chain of the English Center bridge and never varies by more than ten inches from the polygonal geometry of the actual bridge chain.
2. *Rigid bridge towers* — The cable terminates at the top of each tower and these end points are considered to be fixed against any translational movement.
3. *Parabolic cable and floor girder continuously connected by inextensible suspenders* — The cable and girder displace together such that the vertical displacement of the cable and girder are always equal at any vertical section in the span. For example, in Fig. 27, the vertical displacement of point A must always equal the vertical displacement of point B.

The results from both the Elastic and Deflection Theory analyses presented here were obtained using the formulations presented in Steinman's *Practical Treatise*.[99]

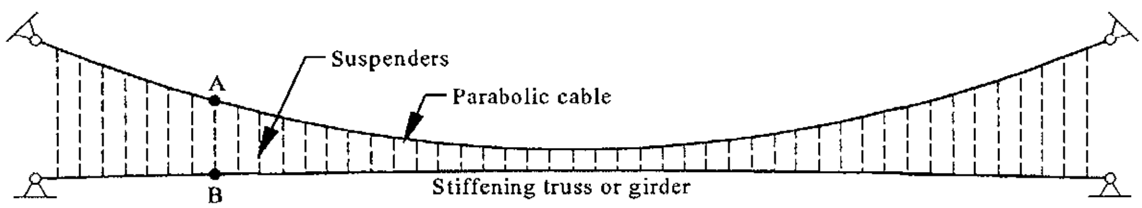

Figure 27 Model B-NL — non-linear version of Model B.

[99] David B. Steinman, *A Practical Treatise on Suspension Bridges,* 1st ed. (New York: John Wiley and Sons, 1922) and 2nd ed. (1929). The first edition contains only the Elastic Theory, while the second edition includes the Deflection Theory as Appendix D.

Table 2 compares the mid-span deflections due to a unit live load of 1 kip placed at the mid-span of the bridge (panel point L6) as determined by linear elastic computer analyses, Elastic Theory, and Deflection Theory. For the Deflection Theory analysis a dead load of 436 lb/ft was used. The mid-span deflections from the computer analysis (1.07 inches) and Elastic Theory (0.89 inches) correspond well, showing that Model B-NL is an acceptable approximation for the geometry of Model B. Because Model B includes the effects of tower and backstay flexibility, it results in a structure with slightly greater overall flexibility, and therefore a larger mid-span deflection, than Model B-NL.

Table 2 Deflection and moment due to 1-kip load at mid-span.

Analysis type	Mid-span deflection (in)	Mid-span moment (kip-in)
Linear elastic (Model B)	1.07	209
Elastic Theory (Model B-NL)	0.89	196
Deflection Theory (Model B-NL)	0.51	135
Ratio Elastic/Deflection	0.57	0.69

Comparing the Elastic and Deflection Theory results in Table 2 reveals that use of the Deflection Theory can result in both smaller deflections and smaller girder moments for the same loading condition. Similar to Navier's theory, the Deflection Theory accounts for the stiffening effects of the dead-load tension in the parabolic cable, resulting in smaller live-load deflections and moments. For the unit load at mid-span on this bridge, the Deflection Theory deflection is 57 percent of that of the Elastic Theory, and the moment 69 percent. The reduction of deflections and moments as determined by the Deflection Theory compared to the Elastic Theory may be substantially greater for asymmetric live loading conditions. Table 3 compares the effects of a unit load at a quarter point (panel point L3 or L9). Here the Deflection Theory deflection is 38 percent of that of the Elastic Theory, and the moment 53 percent. In general, the amount of reduction possible by a Deflection Theory analysis depends on the bridge span, dead load, and girder stiffness. This reduction increases as span length increases, dead load increases, and girder stiffness decreases, although span length is the most significant factor.[100] The development of the Deflection Theory and its application in design in the early twentieth century made possible a rapid increase in suspension bridge spans. The Williamsburg Bridge, built on an Elastic Theory design in 1903, had a 1,600-foot main span. The Benjamin Franklin Bridge, which was in 1926 the first major bridge to be built on a Deflection Theory design, does not demonstrate a dramatic increase in span length at 1,750 feet. Five years later, however, the

[100] Steinman combines the bridge properties of span, dead load, and girder stiffness into a "stiffness factor" and provides charts that relate stiffness factor to percent stress reduction in the girder. See Steinman, *Practical Treatise,* 2nd ed.

Deflection Theory's emphasis on cable stiffness made it possible for the George Washington Bridge's 3,500-foot main span to double this record.[101]

Table 3 Deflection and moment due to 1-kip load at quarter point.

Analysis type	Quarter-point deflection (in)	Quarter-point moment (kip-in)
Linear elastic (Model B)	2.44	293
Elastic Theory (Model B-NL)	2.45	298
Deflection Theory (Model B-NL)	0.92	157
Ratio Elastic/Deflection	0.38	0.53

The history of suspension bridges reveals a wide variety of stiffening methods, (see Fig. 3), of which the English Center bridge is a unique surviving example. Nevertheless, the most common stiffening method remains the deck-stiffening truss with parallel chords used in such bridges as the 1890 Kellams Bridge or the 1904 Riegelsville Bridge.

It is of historical interest to estimate the size of a deck-stiffening truss that would produce approximately the same vertical stiffness as the English Center bridge's diagonal trussing. During load testing, the bridge deflected about 0.72 inches under a truck load of 18.6 kips at mid-span. Analyzing Model B-NL with the Deflection Theory under a 9.3-kip point load (per truss) at mid-span, a longitudinal girder moment of inertia of 16,070 in^4 is required to limit the mid-span deflection to approximately 0.72 inches. This increased moment of inertia is about 17.5 times greater than the existing girder moment of inertia of 918 in^4.

To estimate the size of a truss with this moment of inertia, assume a depth-to-span ratio of 1/50, typical for suspension bridges of the late nineteenth and early twentieth centuries. For the 300-foot span of the English Center bridge, the truss depth would be approximately six feet, measured from the centerline of the upper chord to the centerline of the lower chord. The required upper and lower chord areas, A, for a desired moment of inertia, I, and given truss depth, d, may be calculated from $A=2I/d^2$. The required area of each upper and lower chord is 6.20 in^2. For the 300-foot span of the English Center bridge, the total weight of both chords would be about 12,660 lbs or 42 lb/ft (see Table 4).

[101] See HAER No. NY-128 for documentation of the Williamsburg Bridge, and HAER No. NY-129 for the George Washington Bridge.

Table 4 Estimated weight of stiffening truss required to provide vertical stiffness equal to the English Center bridge as built.

Member	Length per panel (ft)	Number	Total length (ft)	Area (in^2)	Weight (lb)
Upper chord	—	—	300	6.20	6330
Lower chord	—	—	300	6.20	6330
Verticals	6.00	49	294	5.0	5000
Diagonals	8.66	48	416	5.0	7070
				Total weight	24730
				Unit weight (lb/ft)	82.44

The stiffening truss also would have both verticals and diagonals connecting the upper and lower chords. To estimate the weight of the verticals, divide the 300-foot span into forty-eight bays of 6'-3" each, resulting in forty-nine vertical posts each 6'-0" high. If the truss were built with a single diagonal pattern (Pratt or Howe) there would be forty-eight diagonals each approximately 8'-8" long. Estimating the cross sectional area of the verticals and diagonals as about five square inches, then the weight of verticals and diagonals would be 5,000 lbs and 7,070 lbs, respectively. The total weight of the truss is estimated to be about 24,730 lbs or 82.44 lb/ft (see Table 4).

The estimated weight of this stiffening truss should be compared to the weight of the stiffening system of the English Center bridge, as built. The total cross sectional area of the longitudinal floor girder is 10.75 in^2 for a total weight of 10,974 lbs or 36.58 lb/ft, and the total weight of all the diagonals is 3,754 lbs or an average of 12.51 lb/ft (see Table 5). Thus, the total weight of the stiffening system of the English Center bridge is 14,728 lbs or 49.09 lb/ft.

Table 5 Weight of stiffening system of English Center bridge as built.

Member	Total length (ft)	Area (in^2)	Weight (lb)
Floor girder	300	10.75	10974
Diagonals	—	—	3754
		Total weight	14728
		Unit weight (lb/ft)	49.09

Steel weight, although a significant consideration when transporting materials to a rural site, is but one of many factors affecting the cost of a bridge. A designer must also consider fabrication, erection, and other costs when determining the most economical structural form. Nonetheless, the English Center bridge's system of diagonal trussing combined with a relatively flexible longitudinal floor girder uses substantially less material than a conventional deck-stiffening truss of equal overall vertical stiffness.

6. OBSERVATIONS AND CONCLUSIONS

The Lower Bridge at English Center is not of the cantilever type because the girder-to-tower connections were detailed with horizontal slots. It is also not a conventional deck-stiffened suspension bridge because of diagonals between the chain and girder that form a truss. Rather, it is an example of a fully trussed, inverted two-hinged arch. Although no design documents were found, it is likely that the bridge was conceptualized, analyzed, and designed as an inverted trussed arch.

Both uniformly distributed and non-uniform live loads are carried principally by truss action. The erection sequence determined how the bridge's dead load was initially carried. If the falsework was removed before tightening the diagonals, then the dead load was carried as in a deck-stiffened suspension bridge. If the falsework was removed after the diagonals were tightened, then the dead load was carried by inverted trussed arch action. The effects of the original erection sequence are now lost because of flood damage, repairs, and retrofits over the years. The actual forces in the members for the dead load and the tensioning of the diagonals are unknown at present. Therefore the actual behavior of the bridge under moving live loads, which depends on when and if the diagonals go slack, is uncertain.

Analytical influence lines agree with those obtained from experimental load testing. They confirm that there is a potential for force reversals (from tension to compression) in the vertical members, which arises solely from truss action and which cannot occur in conventional deck-stiffened suspension bridges.

Analyses show that the system used for the Lower Bridge at English Center — a fully trussed system with a relatively flexible longitudinal girder — uses substantially less material than a conventional stiffening truss of equal vertical stiffness. Therefore the design of the English Center bridge is an effective, materially efficient alternative to a conventional deck-stiffened suspension form for the (small) 300-foot span.

SOURCES CONSULTED

American Cable Company. *The World's Greatest Suspension Bridge: Philadelphia to Camden.* New York: American Cable Company, 1926.

Ammann, Othmar H. "Possibilities of the Modern Suspension Bridge for Moderate Spans." *Engineering News-Record* 90, No. 25 (21 June 1923): 1072-78.

Aurand, Martin. "South Tenth Street Bridge," Pittsburgh, Pennsylvania. National Register of Historic Places Registration Form, 1986. U.S. Department of the Interior, National Park Service, Washington, D.C.

Bender, Charles. "Historical Sketch of the Successive Improvements in Suspension Bridges to the Present Time." *Transactions of the American Society of Civil Engineers* 1 (1872): 28-43.

Billington, David P. "History and Esthetics in Suspension Bridges." ASCE *Journal of the Structural Division* 103, No. 8 (August 1977): 1655-72.

Birdsall, Blair. "Cable-Stiffened Suspension Bridge Updated." *Engineering News-Record* 150, No. 21 (21 May 1953): 32-39.

______. "A Prophetic Design in an Out-of-the-way Place." *Civil Engineering* 24, No. 9 (September 1954): 574-75.

Bleich, Friedrich. *The Mathematical Theory of Vibration in Suspension Bridges.* Washington, D.C.: U.S. Department of Commerce, Bureau of Public Roads, 1950.

Boyer, W. H., and Irving A. Jelly. "An Early American Suspension Span: Description and Brief Historical Account of a Chain Bridge of 1826." *Civil Engineering* 7, No. 5 (May 1937): 338-40.

Briseghella, Lucia. "Anton Claudio Galateo e I ponti Sospesi a Cavi Dell'inizio del XIX secolo." Tesi de Laurea, Instituto Universitario di Architettura di Venezia, 1996.

Buonopane, Stephen G., and David P. Billington. "Theory and History of Suspension Bridge Design from 1823 to 1940." ASCE *Journal of Structural Engineering* 119, No. 3 (March 1993): 954-77.

Cowper, A. E. "Cowper's Inverted-Arch Suspension Bridge." *Engineering* 3 (22 March 1867): 277.

______. "Railway Suspension Bridge." *The Civil Engineer and Architect's Journal* 10, No. 123 (December 1847): 369.

Delaware River Joint Toll Bridge Commission. *General Information on the Non-Toll Bridges.* Morrisville, Pennsylvania: Delaware River Joint Toll Bridge Commission, 1995.

Delaware River Port Authority. "Bridge Information," September 1998. URL: http://www.drpa.org/brg_info.html.

Engel, Harry J. "300' Span Steel Suspension Bridge Over Little Pine Creek — Inspection Drawing," 24 June 1968. PennDOT Maintenance District 3-2, Montoursville, Pennsylvania.

Fidler, T. Claxton. "Arched-Ribs and Suspension Bridges, with Their Auxiliaries." *Engineering* (London) 20 (5 November 1875): 351-52, (3 December 1875): 429-30, (24 December 1875): 487-88, and (31 December 1875): 509-510; 21 (28 January 1876): 63-64 and (10 March 1876): 183-84.

______. "Suspension Bridges and Arches." *Engineering* (London) 19 (30 April 1875): 372-74.

Finch, J. K. "Wind Failures of Suspension Bridges: or Evolution and Decay of the Stiffening Truss." *Engineering News-Record* 126 (13 March 1941): 402-407.

Gasparini, Dario A., and David Simmons. "American Truss Bridge Connections in the 19th Century, I: 1829-1850." ASCE *Journal of Performance of Constructed Facilities* 11, No. 3 (August 1997): 119-29.

"The Grand Avenue Bridge, St. Louis, Mo." *Engineering News* 24 (6 June 1891): 8-9.

Hopkins, H. J. *A Span of Bridges.* New York: Praeger, 1970.

Jakkula, Arne A. "A History of Suspension Bridges in Bibliographical Form." *Bulletin of the Agricultural and Mechanical College of Texas,* 4th Series, vol. 12, No. 7 (1 July 1941).

Kemp, Emory L. "Links in a Chain: The Development of Suspension Bridges 1801-1870." *The Structural Engineer,* No. 8 (August 1979): 255-63.

A. G. Lichtenstein and Associates. "BMS No. 41-4001-0270-0000, Final Bridge Inspection Report," November 1994. PennDOT Maintenance District 3-2, Montoursville, Pennsylvania.

Maney, G. A. "New Type Suspension Bridge Proposed." *Engineering News-Record* 126 (24 April 1941).

Melan, Josef. "Theorie der eisernen Bogenbrücken und der Hängenbrucken." *Handbuch der Ingenieurwissenschaften,* vol. 2, part 4. Leipzig: Wilhelm Engelmann, 1888.

______. *Theory of Arches and Suspension Bridges.* Trans. David B. Steinman. Chicago: Myron C. Clark, 1913.

Merriman, Mansfield, and Henry S. Jacoby. *A Text-Book on Roofs and Bridges,* part 4, *Higher Structures.* New York: John Wiley and Sons, 1907.

Modjeski and Masters, staff of. "Suspension Bridges and Wind Resistance." *Engineering News-Record* 127 (23 October 1941): 565-68.

Navier, Claude L. M. H. *Rapport a Monsieur Becquey, conseiller d'etat, directeur general des ponts et chausees et des mines; et mémoire sur les ponts suspendus.* Paris: Imprimérie Royal, 1823.

Perez, Carlos Renc. To Dario A. Gasparini, 23 July 1998.

Peters, Tom F. *The Development of Long-Span Bridge Building.* Zurich: Verlag der Fachüreine, 1979.

Plowden, David. *Bridges: The Spans of North America.* New York: Norton, 1974.

Podlony, Walter, Jr., and John F. Fleming. "Historical Development of Cable-Stayed Bridges." ASCE *Journal of the Structural Division* 98, No. 9 (September 1972): 2079-95.

Pugsley, Alfred G. *The Theory of Suspension Bridges.* London: Edward Arnold, 1968.

Rankine, William. *A Manual of Applied Mechanics.* 5th cd. London: Charles Griffin and Company, 1869.

John A. Roebling's Sons Company. *A Century of Progress.* Trenton, New Jersey: John A. Roebling's Sons Company, 1931.

Sayenga, Don. Draft of report on wirc-cable suspension bridges, 1998. U.S. Department of the Interior, Historic American Engineering Record, Washington, D.C.

Sehuyler, Hamilton. *The Roeblings: A Century of Engineers, Bridge-builders, and Industrialists.* Princeton: Princeton University Press, 1931.

Seguin, Marc. *Des ponts en fil de fer.* Paris: Bachelier, 1824.

Simmons, David A. "Bridge Building on a National Scale: The King Iron Bridge and Manufacturing Company." *IA: The Journal of the Society for Industrial Archeology* 15, No. 2 (1989): 23-39.

Sollenberger, N. J. "'Cable Truss' Design Greatly Increases Stiffness." *Civil Engineering* 24, No. 9 (September 1954): 576-79.

Steinman, David B. *A Practical Treatise on Suspension Bridges.* 1st ed. New York: John Wiley and Sons, 1922.

______. *A Practical Treatise on Suspension Bridges.* 2nd ed. New York: John Wiley and Sons, 1929.

______. "The Reconstruction of the Brooklyn Bridge." *Columbia Engineering Quarterly* (November 1952).

"Suspension Footbridge." *Engineering News-Record* 140 (26 February 1948): 9.

Tyrrell, Henry G. *History of Bridge Engineering.* Chicago, 1911.

U.S. Department of the Interior, Historic American Engineering Record (HAER), No. PA-461, "Lower Bridge at English Center," 1997. Prints and Photographs Division, Library of Congress, Washington, D.C.

______, HAER No. PA-470, "Kellams Bridge," 1997. Prints and Photographs Division, Library of Congress, Washington, D.C.

______, HAER No. PA-490, "Three Sisters Bridges," 1998. Prints and Photographs Division, Library of Congress, Washington, D.C.

White, Anthony G. *Bridge Architecture and Design: Suspension Bridges — a Selected Bibliography.* Monticello, Illinois: Vance Bibliographies, 1981.

Zink, Clifford W., and Dorothy W. Hartman. *Spanning the Industrial Age: The John A. Roebling's Sons Company.* Trenton, New Jersey: Trenton Roebling Community Development Corporation, 1992.

APPENDIX A: PARTIAL LIST OF SUSPENSION BRIDGES IN PENNSYLVANIA

Year Compl.	Spanning	Name and/or Location (Extant bridges in bold type)	Main Span Length(s)	Type	Designer	Ref.
1801	Jacob's Cr.	Uniontown-Greensburg Turnpike	70'	chain	James Finley	a, b, c
c. 1810		Brownsville	112'	chain	James Finley	c, d
c. 1810		Brownsville	120'	chain	James Finley	c, d
1809	Neshammy (?) Cr.	Bucks County	2@100'	chain	James Finley	d
1809	Schuylkill R.	Fairmount Park (Philadelphia)	2@153'	chain	John Templeman (Finley patent)	b, c
1811	Schuylkill R.	Fairmount Park (Philadelphia)	2@153'	chain	John Templeman (?)	b, c
1811	Lehigh R.	Northampton	2@100'	chain	James Finley	b, c, e
1814	Lehigh R.	Northampton	2-1/2 spans, 475' total	chain	James Finley	d
1815	Lehigh R.	Allentown	2@230'	chain	James Finley	b, c, e
1816	Schuylkill R.	Fairmount Park (Philadelphia)	408'	cable	Josiah White and Erskine Hazard	b, c, e
1826	Lehigh R.	Lehigh Gap	160'	chain	Jacob Blumer (Finley patent)	f
1842	Schuylkill R.	Callowhill Street (Philadelphia)	358'	cable	Charles Ellet	b, c, e
1845	Allegheny R.	Allegheny Aqueduct (Pittsburgh)	7@162'	cable	John A. Roebling	b, c, e
1847	Monongahela R.	Smithfield Street (Pittsburgh)	8@188'	cable	John A. Roebling	b, c, e
1848	Delaware R.	**Delaware Aqueduct** (Lackawaxen)	4@131'-142'	cable	John A. Roebling	b
1848	Lackawaxen R.	Lackawaxen Aqueduct	2@115'	cable	John A. Roebling	b, d
1853	Delaware R.	Easton			John Murphy	d
1857	Lehigh R.	Glendon	Two spans	cable	Edwin A. Douglas	g
1860	Allegheny R.	Sixth Street (Pittsburgh)	2@344'	cable	John Roebling	b, e
1866	Allegheny R.	Oil City	2@325', 1@162'-6"	cable		h
1866	Susquehanna R.	Market Street (Williamsport)	5@200'	cable	John Murphy (?)	i
1870*	Delaware R.	Lordville-Equinunk	345'	chain		e
1871	Allegheny R.	Warren	470'	cable		d, e
1876	Monongahela R.	Point Bridge (Pittsburgh)	800'	chain	Edward Hemberle	b, c, e
1876	Allegheny R.	Oil City	500'	cable	Charles Roebling	c, e, j

Year Compl.	Spanning	Name and/or Location (Extant bridges in **bold type**)	Main Span Length(s)	Type	Designer	Ref.
1884	Allegheny R.	Seventh Street (Pittsburgh)	2@330'	chain	Gustav Lindenthal	c, e
1888	Lehigh R.	Mauch Chunk (Jim Thorpe)	360' (?)	cable (?)	William Hildenbrand	d
1889	Delaware R.	**Kellams Bridge** (Stalker)	388'	cable	David Kellam	k
1891	Little Pine Cr.	**Lower Bridge** (English Center)	300'	chain	Dean & Westbrook	l
1891	Little Pine Cr.	Upper Bridge (English Center)	400' (?)	chain	Dean & Westbrook	l
1897	Ohio R.	Rochester	800'	cable	E. K. Morse	d
1900	Lehigh R.	Footbridge (Easton)	2@279'	cable	Henry G. Tyrrell	c, e
1904	Delaware R.	**Riegelsville**	3@186'-200'	cable	John A. Roebling's Sons Co.	m
1926	Delaware R.	**Ben Franklin** (Philadelphia)	1750'	cable	Ralph Modjeski	n
1926	Allegheny R.	**Ninth Street** (Pittsburgh)	995'	chain	Allegheny County	o
1926	Allegheny R.	**Seventh Street** (Pittsburgh)	1061'	chain	Allegheny County	o
1928	Allegheny R.	**Sixth Street** (Pittsburgh)	995'	chain	Allegheny County	o
1933	Monongahela R.	**South Tenth Street** (Pittsburgh)	725'	cable	Vernon Covell	p
1947	Delaware R.	**Footbridge** (Lumberville)	4@146'-157', 1@85'	cable	John A. Roebling's Sons Co.	q
1957	Delaware R.	**Walt Whitman** (Philadelphia)	2000'	cable	Othmar Ammann	b, r
no date		Franklin		cable		c, d
no date		Island Park (Easton)	Two spans	cable		c, d
no date	Youghiogheny R.	McKeesport	320'	cable (?)		d
no date	Penn'a Railroad	Philadelphia		cable (?)		d
no date	Schuylkill R.	Union Bridge (Philadelphia)	3@191'			d
no date		Penn Public Service Corp. (Rockwood)		cable	John A. Roebling's Sons Co.	s

* *Historic Highway Bridges of Pennsylvania* (Harrisburg: Pennsylvania Department of Transportation, 1986), 87, lists an eye-bar bridge spanning 347 feet and built in 1904. The bridge has unfortunately been demolished.

Sources:

a. J. K. Finch, "Wind Failures of Suspension Bridges: or Evolution and Decay of the Stiffening Truss," *Engineering News-Record* 126 (13 March 1941): 74.

b. David Plowden, *Bridges: The Spans of North America* (New York: Norton, 1974).

c. Henry G. Tyrrell, *History of Bridge Engineering* (Chicago, 1911).

d. Arne A. Jakkula, "A History of Suspension Bridges in Bibliographical Form," *Bulletin of the Agricultural and Mechanical College of Texas*, 4th Series, vol. 12, No. 7 (1 July 1941).

e. David B. Steinman, *A Practical Treatise on Suspension Bridges*, 2nd ed. (New York: John Wiley and Sons, 1929), Appendix E.

f. W. H. Boyer and Irving A. Jelly, "An Early American Suspension Span," *Civil Engineering* 7, No. 5 (May 1937): 338-40.

g. Don Sayenga, draft of report on wire-cable suspension bridges, 1998 (U.S. Department of the Interior, Historic American Engineering Record, Washington, D.C.).

h. "Bridge Across Allegheny," *American Railway Journal* 38, No. 1528 (28 July 1865), cited in Jakkula, "A History of Suspension Bridges," 180.

i. Alfred P. Boller, "Williamsport Suspension," *Journal of the Franklin Institute* 5, No. 4 (April 1866): 217-219, cited in Jakkula, "A History of Suspension Bridges," 179-80.

j. Hamilton Schuyler, *The Roeblings: A Century of Engineers, Bridge-Builders and Industrialists* (Princeton: Princeton University Press, 1931), 312.

k. U.S. Department of the Interior, Historic American Engineering Record (HAER), No. PA-470, "Kellams Bridge," 1997, Prints and Photographs Division, Library of Congress, Washington, D.C.

l. U.S. Department of the Interior, Historic American Engineering Record (HAER), No. PA-461, "Lower Bridge at English Center," 1997, Prints and Photographs Division, Library of Congress, Washington, D.C.

m. Delaware River Joint Toll Bridge Commission, "Riegelsville Non-Toll Bridge," *General Information on the Non-Toll Bridges* (Morrisville, Pennsylvania: Delaware River Joint Toll Bridge Commission, 1995), 3.

n. American Cable Company, *The World's Greatest Suspension Bridge: Philadelphia to Camden* (New York: American Cable Company, 1926).

o. U.S. Department of the Interior, Historic American Engineering Record (HAER), No. PA-490, "Three Sisters Bridges," 1998, Prints and Photographs Division, Library of Congress, Washington, D.C.

p. Martin Aurand, "South Tenth Street Bridge," Pittsburgh, Pennsylvania. National Register of Historic Places Registration Form, p. 1, 1986, U.S. Department of the Interior, National Park Service, Washington, D.C.

q. Delaware River Joint Toll Bridge Commission, "Lumberville-Raven Rock Non-Toll Pedestrian Bridge," *General Information on the Non-Toll Bridges* (Morrisville, Pennsylvania: Delaware River Joint Toll Bridge Commission, 1995), 3.

r. Delaware River Port Authority, "Bridge Information," September 1998, URL: http://www.drpa.org/brg_info.html.

s. John A. Roebling's Sons Company, *A Century of Progress* (Trenton, New Jersey: John A. Roebling's Sons Company, 1931).

APPENDIX B: LINEAR ANALYSIS DATA

Table B-1 Member section properties of Lower Bridge at English Center.

Section	Area, A (in^2)	Depth, d (in)	Width, b (in)	Moment of inertia		Section modulus	
				In plane, I_X (in^4)	Out of plane, I_Y (in^4)	In plane, S_X (in^3)	Out of plane, S_Y (in^3)
Lower chord	10.75	24.00	6.25	918.47	10.32	76.54	3.30
Upper chord	14.00	4.00	3.50	18.67	0.89	9.33	0.51
Box vertical	2.86	12.25	12.25	89.35	89.35	14.59	14.59
H vertical	4.75	6.25	8.38	10.29	66.39	3.29	15.85
Tower, seg. A	19.61	47.42	16.00	10107.	530.41	426.24	66.30
Tower, seg. B	19.61	38.50	16.00	6533.	530.41	339.37	66.30
Tower, seg. C	19.61	21.01	16.00	1793.	530.41	170.65	66.30
Diagonal, 7/8" dia.	0.88	No bending properties defined.					
Diagonal, 1" dia.	1.57	No bending properties defined.					
Diagonal, 1 1/8" dia.	1.99	No bending properties defined.					
Diagonal, 1 1/4" dia.	2.45	No bending properties defined.					

Table B-2 Dead loads for deck components, typical for Models A through H.

Member	Length per panel (ft)	Unit weight (lb/ft)	Weight per panel (lb)	Weight per truss (lb)	x 1.05 for misc. steel (lb)
Floor beam	3@18.0 = 54.0	40.80	2203.2	1101.6	
Stringer (channel)	2@25.0 = 50.0	9.80	490.0	245.0	
Stringer (1-beam)	6@25.0 = 150.0	15.30	2295.0	1147.5	
Deck (19.3 psf, 15.5 ft wide)	25.0	299.15	7478.8	3739.4	
Wheel guard	2@25.0 = 50.0	14.90	745.0	372.5	
Guide rail post	6@4.0 = 24.0	25.50	612.0	306.0	
Guide rail	2@25.0 = 50.0	8.77	438.5	219.3	
Subtotal, excluding lateral bracing				7131.2	7487.8
Lateral bracing A (panels 1, 2, 11, 12)	2@21.4 = 42.8	7.20	308.5	154.3	
Lateral bracing B (panels 3, 4, 9, 10)	2@21.4 = 42.8	6.10	261.4	130.7	
Lateral bracing C (panels 5, 6, 7, 8)	2@21.4 = 42.8	4.90	210.0	105.0	
Subtotal + A = Total at L1, L11				7285.5	7649.8
Subtotal + 0.5A + 0.5B = Total at L2, L10				7273.7	7637.4
Subtotal + B = Total at L3, L9				7261.9	7625.0
Subtotal + 0.5B + 0.5C = Total at L4, L8				7249.1	7611.5
Subtotal + C = Total at L5, L6, L7				7236.2	7598.0

Table B-3 Dead loads, Models A through H.

Model	Dead load* (kips) applied to panel point:					
	L1, L11	L2, L10	L3, L9	L4, L8	L5, L7	L6
A, D, F, G**	10.48	10.54	10.44	10.35	10.35	10.36
B	10.24	10.26	10.11	10.00	9.93	9.91
E	10.31	10.51	10.42	10.31	10.12	9.93
H	10.18	10.18	10.19	10.18	10.23	10.26
Model	Dead load* (kips) applied to panel point:					
	L1, L12	L2, L11	L3, L10	L4, L9	L5, L8	L6, L7
C	7.68	10.17	10.12	10.07	10.05	10.06

Notes:

* Dead load includes weight of one truss, times 1.05 for miscellaneous steel, plus deck loads from Table B-2.

** It is assumed that the forms represented by models D, F, and G would be made to resemble Model A, with "dummy" members of equivalent weight disguising their structural hinges. As a case in point, James Madison Porter III added superfluous eye-bars to conceal the Northampton Street Bridge's suspended span.

APPENDIX C: LOAD TESTING DATA

C.1. Strain-Measuring Devices

Three types of strain-measuring instruments were applied to selected members of the upstream (east) truss for the load test:

1. *BDI Strain Transducers* — Ten specialty strain transducers manufactured by Bridge Diagnostics, Incorporated (BDI), Boulder, Colorado, were used to measure strains in vertical U1-L1, diagonals U0-L1 and U5-L6, and segment L0-L1 of the stiffening girder. These transducers incorporate strain gages in a Poisson full-bridge configuration. The transducers were calibrated by the manufacturer prior to shipment, and a calibration factor was furnished for each unit. The serial numbers were recorded as the instruments were installed.

2. *Direct Current Linear Variable Differential Transformers (DCLVDTs)* — Lucas Schaevitz model GCD-125-050 DCLVDTs, set with an approximate gauge length of four inches, were placed on the links of the eye-bar chain as close to the towers as feasible (just above U2), and near mid-span (between U5 and U6). The DCLVDT mounting blocks were clamped to the eye-bars using a large C-clamp, and the spring-loaded push rods for the DCLVDTs were set up to bear on a light-gauge steel angle, also clamped to the eye-bar. At each location, one transducer was installed on each of four eye-bars. The DCLVDTs used in this test had a nominal calibration factor of 0.005 inches per volt of output signal, and a resolution of close to 10^{-5} inches. These transducers measure relative displacement between two points, which can be converted to strain units by dividing by the length between the two clamping points, or gage length.

3. *Bonded Electrical Resistance Strain Gages* — Two 6-millimeter-long, 120-ohm foil strain gages were installed in a 1/4 bridge configuration on two of the eye-bars of the backstay at the anchorage location. These gages are bonded to the steel (after removing the paint) and respond with the same strain as the steel. As the foil grid strains, its electrical resistanee changes. The changes in resistance can be measured as a change in the voltage output for a given input voltage.

C.2. Data Acquisition

The BDI strain transducers were connected to an on-board excitation and bridge amplifier manufactured by IOTech, fully compatible with the data acquisition system. The excitation voltage was 5.00 volts, and the signal was amplified by a gain factor of 100. The resulting calibration of amplified signal to strain is given in Table C-1. The output signal from the DCLVDTs was taken directly by the data acquisition system; gage lengths and calihrations are listed in Table C-2. The strain gages were connected to a Vishay 2120 bridge amplifier. An approximate excitation of 2 volts was applied to the strain gages, which were calibrated to a signal of 1 millivolt per microstrain (mV/με), as shown in Table C-3. For the static tests, the channels were sampled at a rate of 20 Hz for a duration of 60 seconds.

Table C-1 Strain transducers placed on Lower Bridge at English Center.

Member	Serial No.	Nominal Calibration factor ($\mu\epsilon/mV/V_{exc}$)	Gain	Calibration ($\mu\epsilon/V$)
U1-L1*	3781	518.6	100	1037
	3782	486.8	100	974
	3883	508.5	100	1017
	3784	549.0	100	1098
U0-L1	3785	551.7	100	1103
	3786	541.4	100	1082
L0-L1	3787	471.5	100	943
	3788	501.7	100	1003
U5-L6**	3789	476.1	100	952
	3805	541.2	100	1082

Notes:

* These strain transducers were moved to the floor system for subsequent test series not described in this report.

** These strain transducers were initially attached to the stiffening girder between L5 and L6, but the amplifier for No. 3805, which was on the bottom flange, failed. No usable data were obtained from that transducer. Since the stiffening girder was subjected to large bending moment, it was not possible to infer the axial force in that member. These instruments were moved to U5-L6 for the test series described in this report, and again to U6-L5 for subsequent series.

Table C-2 DCLVDT instruments placed on Lower Bridge at English Center.

Member	Serial No.	Nominal Calibration factor (in/V)	Gage Length (in)	Calibration (με/V)
U1-U2	22890	0.005	3.50	1428
	22900	0.005	3.50	1428
	15808	0.005	3.50	1428
	22907	0.005	4.0	1250
U5-U6	22893	0.005	4.0	1250
	22880	0.005	4.0	1250
	15815	0.005	3.5	1428
	15816	0.005	4.0	1250

Table C-3 Bonded strain gages placed on Lower Bridge at English Center.

Member	Calibration (με/V)
Backstay	1000
	1000

C.3. Load Testing Results

Table C-4 shows the axial force data used to plot experimental influence lines.

Table C-4 Axial force data from load testing, Lower Bridge at English Center.

Distance of 1-kip load from L0 (ft)	Force (kips) in member:						Distance of 1-kip load from L0 (ft)	Force in member U5-L6 (kips)
	U1-L1	U0-L1	L0-L1	U1-U2	U5-U6	Backstay		
							-45.00	0.01
-25.00	0.01	-0.19	-0.14	-0.03	-0.10	-0.04	-13.00	0.02
7.00	0.03	0.00	0.01	0.05	-0.12	-0.02	-2.00	-0.04
18.00	0.00	0.54	-0.15	0.07	0.10	0.14	13.00	0.00
33.00	-0.07	0.63	-0.50	-0.10	0.13	0.20	25.00	0.02
45.00	-0.15	0.64	-0.44	-0.08	0.11	0.33	38.00	-0.04
58.00	-0.25	0.52	-0.21	0.11	0.05	0.53	52.00	-0.02
72.00	-0.25	0.36	-0.24	0.30	0.05	0.60	63.00	-0.03
83.00	-0.18	0.25	-0.27	0.38	-0.02	0.72	75.00	-0.11
95.00	-0.08	0.09	-0.32	0.49	-0.12	0.77	88.00	0.00
108.00	-0.04	-0.07	-0.13	1.02	-0.17	0.91	100.00	-0.18
120.00	0.02	-0.16	-0.15	1.04	-0.54	0.87	113.00	0.01
133.00	0.05	-0.25	-0.20	1.37	-0.50	0.93	125.00	0.39
145.00	0.07	-0.33	0.03	1.15	-0.69	0.72	138.00	0.30
158.00	0.06	-0.33	0.00	1.02	-0.30	0.70	163.00	0.22
183.00	0.06	-0.31	-0.01	0.82	0.18	0.60	188.00	0.09
208.00	0.05	-0.31	0.08	0.72	0.10	0.41	213.00	0.08
233.00	0.02	-0.27	0.02	0.57	0.37	0.27	238.00	0.06
258.00	0.04	-0.29	-0.07	0.36	0.23	0.13	262.00	0.06
282.00	0.00	-0.18	-0.12	0.32	0.05	0.10	280.00	0.06
300.00	0.03	-0.17	-0.23	0.18	-0.16	0.11	305.00	0.04
325.00	0.04	-0.25	-0.20	0.10	-0.11	0.09		

ADDENDUM TO:
LOWER BRIDGE AT ENGLISH CENTER
Spanning Little Pine Creek at State Route 4001
English Center
Lycoming
Pennsylvania

HAER PA-461
PA,41-ENGCE,1-

FIELD RECORDS

HISTORIC AMERICAN ENGINEERING RECORD
National Park Service
U.S. Department of the Interior
1849 C Street NW
Washington, DC 20240-0001